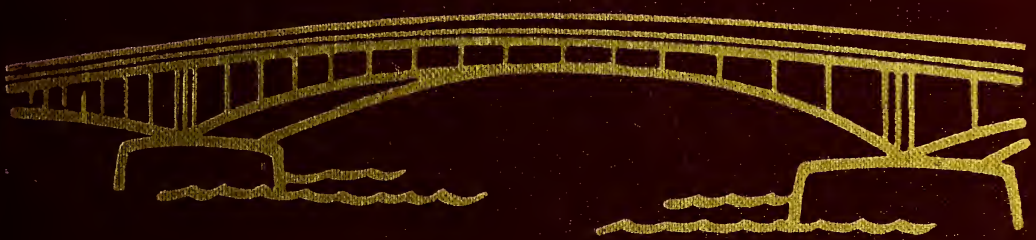
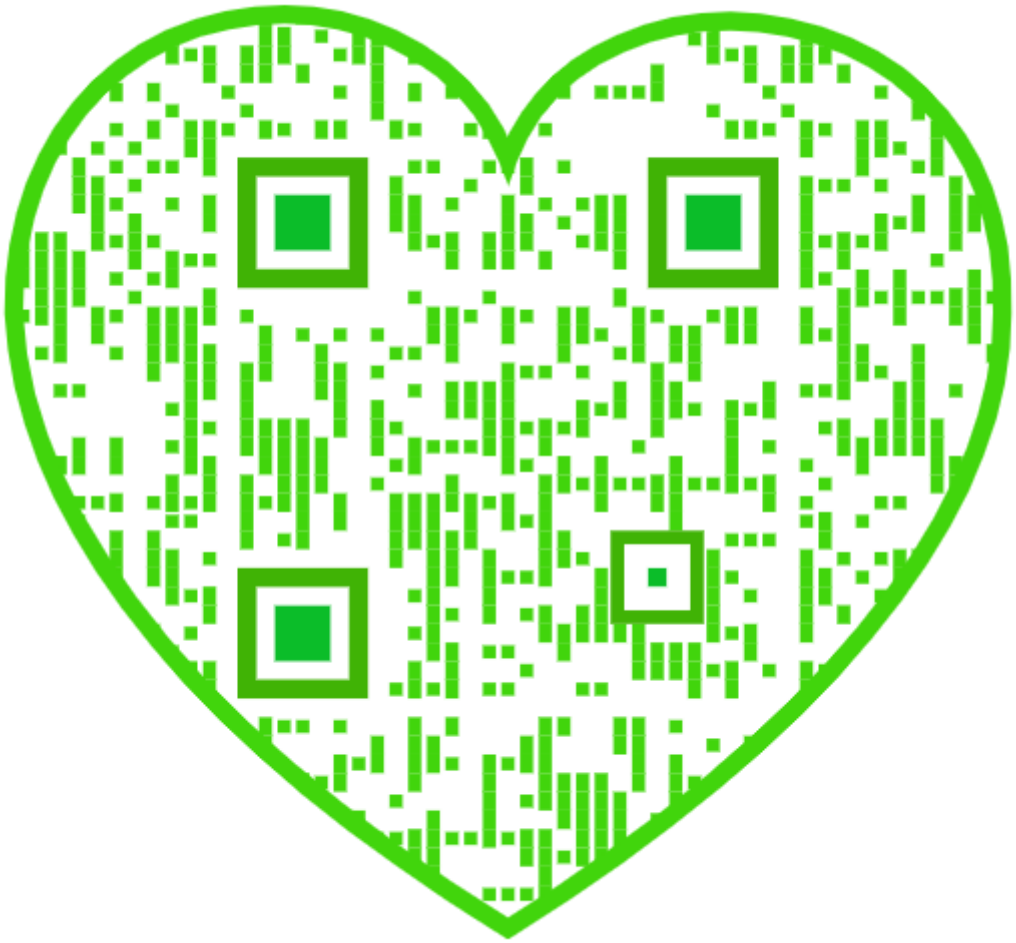
Comparative Bridge Designs



The James F. Lincoln Arc Welding Foundation



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COMPARATIVE BRIDGE DESIGNS

Editor

James G. Clark

Professor of Civil Engineering

University of Illinois

54-29309

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CONTENTS

Pages

Preface V

Contents Page VII

Chapter I—INTRODUCTION

The James F. Lincoln Arc Welding Foundation 1

The Award Programs for Design of Bridges 1

The 1952 Awards 7

Summary of the Designs 9

Chapter II—CONTINUOUS BEAMS AND GIRDERS

Continuous Girder Bridges Without Floorbeams 11

Continuous Girder Bridges With Floorbeams 64

Continuous Girder Bridges—Prestressed 84

Chapter III—SIMPLE BEAM SPANS 104

Chapter IV—SIMPLE TRUSSES 116

Chapter V—CONTINUOUS TRUSSES 164

Chapter VI—ARCHES 175

CHAPTER I

INTRODUCTION

The James F. Lincoln Arc Welding Foundation

Since its creation in 1936, The James F. Lincoln Arc Welding Foundation has been devoted to its object and purpose as stated in the Deed of Trust, "to encourage and stimulate scientific interest in, and scientific study, research and education in respect of, the development of the arc welding industry through advance in the knowledge of design and practical application of the arc welding process".

The activities of the Foundation have included along with other valuable projects, the sponsorship of many programs of various kinds. Among some of the more recent programs have been three that were intentionally limited to the designs of welded highway bridges. The first two, entitled "Welded Bridges of the Future", were conducted in 1949 and 1950 respectively. The third, entitled "Welded Bridges for Steel Conservation", was sponsored in 1952.

The Foundation has provided funds to many libraries for the purchase of books on welding and, consistently, the Trustees of the Foundation have approved the publication of a number of books on welding. "Welded Deck Highway Bridges" was published to make available the more valuable information from the 1949 Award Program and accordingly, "Welded Highway Bridge Design" for the 1950 Award Program. This book presents selected material from some of the designs of the 1952 Award Program.

The Trustees and Officers of the Foundation are:

Trustees:
E. E. Dreese, Chairman, Columbus, Ohio
H. R. Harris, Cleveland, Ohio
T. V. Koykka, Cleveland, Ohio
Officers:
A. F. Davis, Secretary, Cleveland, Ohio
C. G. Herbruck, Assistant Secretary, Cleveland, Ohio

The Award Programs for Design of Bridges

All three bridge programs have been directed toward the task of encouraging and stimulating structural engineers to design welded bridges that will cost less and save steel. The first two

programs stressed the item of low cost, but this 1952 program had as its primary objective the design of bridges requiring less steel.

The 1949 and 1950 programs were alike in that each specified a definite span, width, and loading for the bridge designs entered. The 1949 program concerned structures described as follows: "A two-lane deck highway bridge supported on two end piers 120 feet apart, centerline to centerline of bearings." The 1950 program gave the following description for the bridges: "A Two-Lane Through Highway Bridge With a Span of 250 Feet." Both programs specified a roadway width of 26 feet between the inside faces of the curbs and an H 20-44 loading.

This 1952 Award Program did not limit the span or width of the bridges in the entries; however, it did require two highway bridge designs—one called Exhibit A and the other, Exhibit B. These were described in the Rules and Conditions as follows:

"Exhibit A shall be a design of a modern riveted highway bridge, either a new original design or one of a bridge recently built or designed. If Exhibit A is for a bridge already built or designed, it does not have to be a design of the participant. It may be the design of another engineer, group of engineers, or department, provided the participant has the proper permisison to use it in this Program.

"Exhibit B shall be an all-welded design of a highway bridge prepared by the participant within the period of the competition.

"Both Exhibit A and Exhibit B must be designed for the same span (or spans if continuous), for the same loading conditions (including similar type of floor), for the same number of lanes which must be two or more, of similar structural types and general outline and dimensions, and in compliance with Specifications of this Program. The span (or spans if continuous), highway loading, and type of structure for the entry are not restricted except that the conditions for Exhibit A and Exhibit B must be the same."

With such a classification for the bridge, each participant was allowed to select the type he wished to present. His riveted bridge was to be as light in weight as the design specifications permitted, therefore, his welded bridge, designed to perform the same services, would show the weight-saving advantages accruing from the welded design.

In order that the entries would be in accordance with common bridge design practice, both exhibits were limited to the use of shapes and sizes of material that were currently available. Also, both were to comply with the Specifications of the Rules and

Conditions:

"Inasmuch as Exhibit A and Exhibit B cover bridges of similar structural types designed for the same span (or spans if continuous) and the same highway loading, the substructure for each would be essentially the same. Therefore, the design of the substructure is not included in this competition, but the participant shall show the outline of its upper parts (or other parts to which the superstructure is attached) in the general drawings (plans, sections, and elevations) of the superstructure.

"A floor (curbs and railings) which serves only as such and does not participate otherwise in the strength of the bridge need be designed only so far as is necessary to indicate the dimensions from which the gross weight is determined. A floor (curbs and railings) which is designed to contribute otherwise to the strength of the bridge shall be designed in sufficient detail to indicate the extent and manner of its participation.

"If Exhibit B has a participating floor, Exhibit A must have a floor that participates in a similar manner (if such a floor would save weight). However, if Exhibit A is the design of a bridge that already has been built without a participating floor, the exhibitor shall include the calculations necessary to show the net amount of steel that could have been saved if the bridge had been designed with a participating floor similar to the one he uses in Exhibit B.

"If a participating floor does not save weight for Exhibit A and if Exhibit B has a participating floor, then Exhibit B having a participating floor shall be compared with Exhibit A having a floor that does not participate.

"The bridge shall be designed for either ASTM A 7-46 steel or ASTM A 242-46 steel. If a participant uses ASTM A 242-46 steel, it will be assumed that the fabricator will obtain a steel meeting this specification that is weldable. Exhibit A and Exhibit B must use the same steel and both exhibits must utilize only the shapes and sizes that are currently available. Shapes and sizes as shown in Manual of the American Institute of Steel Construction, Fifth Edition, will be considered currently available for the purpose of this Program.

"For information and guidance, the participants should refer to the 1949 edition of the American Association of State Highway Officials *Standard Specifications for Highway Bridges* and the 1947 edition of the *Standard Specifications for Welded Highway and Railway Bridges* of the American Welding Society.

"For Exhibit A and Exhibit B, the article numbers (of the Ameri-

can Association of State Highway Officials Specifications) shown in parentheses below are to be considered as an integral part of the Specifications of this Program.

FOR EXHIBIT A

Clearances (3.1.8)
Dead Load (3.2.2)
Highway Loading (3.2.5 to 3.2.9 incl.)
Impact (3.2.12)
Longitudinal Forces (3.2.13)
Wind Loads (3.2.14)
Thermal Forces (3.2.15)
Distribution of Loads (3.3.1, 3.3.2, 3.3.5)
Unit Stresses (3.4.1 to 3.4.7 inclusive, 3.4.9, 3.4.10)
Structural Steel Design (3.6.2 to 3.6.53 inclusive, and 3.6.56 to 3.6.106 inclusive)

FOR EXHIBIT B

Except that welding is permitted for all structural members, the above specifications for Exhibit A shall apply. The allowable unit stresses for effective area of weld shall be as given in the American Welding Society's *Standard Specifications for Welded Highway and Railway Bridges,* 1947. If a participant uses ASTM A 242-46 steel, it will be assumed for the purpose of this Program that Part II of Section 2 of these welding specifications applies to ASTM A 242-46 steel, except that the values given there in Table I for members connected by fillet or plug welds shall be as shown, but the values given there in Table I for members connected by butt welds may be increased by the ratio of the base stress for A 242 steel to the base stress for A 7 steel.

The Jury of Award, in rating the merits of entries, gave consideration to the quality of the design of both Exhibit A and Exhibit B, to the proportionate steel savings of Exhibit B as compared with Exhibit A, and the comparative total cost including erection.

The Rules Committee consisted of the following men:

James G. Clark, Chairman
Professor of Civil Engineering
University of Illinois
Urbana, Illinois

E. E. Dreese
Chairman, Board of Trustees, Lincoln Foundation
Chairman, Department of Electrical Engineering
The Ohio State University
Columbus, Ohio

Charles E. Andrew
Chief Consulting Engineer
Washington Toll Bridge Authority
Olympia, Washington

Raymond Archibald
Chairman, Bridge Committee, AASHO
U. S. Bureau of Public Roads
1270 Flood Building
San Francisco, California

R. N. Bergendoff
Consulting Engineer
Howard, Needles, Tammen & Bergendoff
1805 Grand Ave.
Kansas City, Missouri

Harry C. Boardman
Director of Research
Chicago Bridge and Iron Company
Chicago, Illinois

H. W. Brinkman
Vice President & Chief Engineer
Phoenix Bridge Company
Phoenixville, Pennsylvania

E. S. Elcock
Bridge Engineer
State Highway Commission
Topeka, Kansas

N. B. Garver
Consultant
Arkansas State Highway Department
Little Rock, Arkansas

Morris Goodkind
Director and Chief Bridge Engineer
State Highway Department
Trenton, New Jersey

Shortridge Hardesty
Consulting Engineer
Hardesty and Hanover
101 Park Avenue
New York, New York

John I. Parcel
Consulting Engineer
Sverdrup & Parcel, Inc.
1118 Syndicate Trust Building
St. Louis, Missouri

Thomas C. Shedd
Professor of Structural Engineering
University of Illinois
Urbana, Illinois

C. Earl Webb
Chief Engineer
American Bridge Company
Pittsburgh, Pennsylvania

John F. Willis
Engineer of Bridges & Structures
State Highway Department
Hartford, Connecticut

The Jury of Award was composed of five members of the Rules Committee, plus Messrs. Erickson and Hanson.

James G. Clark, Chairman

H. W. Brinkman

E. E. Dreese

E. L. Erickson
Chief, Bridge Branch
Bureau of Public Roads
Washington, D. C.

N. B. Garver

W. E. Hanson
Engineer of Bridge and Traffic Structures
Illinois Division of Highways
Springfield, Illinois

Thomas C. Shedd

The 1952 Awards

Each paper submitted was examined by every member of this Jury before the entire membership of the jury started their meetings to study and discuss the papers as a group. This method of judging was consistent with the extreme precautions taken by the Officers of the Foundation to assure that each exhibit should receive fair and proper consideration as well as confidential handling.

The authors of the thirteen entries which received awards were paid a total of $16,100. A list of the award winners with a very brief description of each bridge is as follows:

First Award—$7267

Elwyn H. King, 2463 Vuelta Grande, Long Beach 15, California. Design—A seven span deck girder bridge composed of cantilevers with suspended spans. Total length—762 ft. Two - 36 ft. roadways, each supported on 5 lines of girders. Welded bridge uses 24 per cent less steel.

Second Award—$3635

Kiser E. Dumbauld, 1844 Chatfield Road, Columbus 12, Ohio. Design—An eight span deck girder bridge having a total length of 798 ft. Two - 26 ft. roadways, two - 6 ft. sidewalks and a 4 ft. median. Ten lines of haunched girders. Steel savings about 26 per cent.

Third Award—$2078

Thomas C. Kavanagh, New York University, College of Engineering, New York 53, New York and Leo Coff, 198 Broadway, New York 38, New York.

Design—A three span continuous deck girder bridge, 990 ft. long, carrying a 36 ft. roadway. The slab and girders of composite construction and prestressed by cables. Adjacent to piers, the girders are box-type structures. Savings in structural steel is 20 per cent.

Ten Honorable Mention Awards—$312 each

Milton D. Randall and Farland Bundy, both of Bridge Division, Texas Highway Department, Austin, Texas.

Design—A three span continuous deck girder bridge, 330 ft. long, having a 28 ft. roadway. Expanded beams between piers, haunched welded girders at piers.

M. O. Elkow, 75 Lee Avenue, Yonkers 5, New York.

Design—A three span continuous deck girder bridge with spans of 160 ft., 200 ft., and 160 ft. respectively. Two - 26 ft. roadways. Two girders 9 ft. deep, 42 ft. apart.

Horace O. Titus, 822 West Second Avenue, Cheyenne, Wyoming.

Design—A three span continuous deck girder bridge having a total length of 165.5 ft. Four girders support the 30 ft. roadway.

Nan-sze Sih, 157-16 20th Road, Whitestone, Long Island, New York.

Design—A deck truss bridge of three continuous spans with a total length of 360 ft. Two trusses 28 ft. apart support a 36 ft. roadway. Composite action between slab and truss top chord.

James H. Jennison, 1612 Coolidge Avenue, Pasadena 7, California.

Design—A deck truss with a span of 150 ft. Composite action between truss top chord and concrete slab. 26 ft. roadway.

Alan R. Cripe, 2901 Hampton Road, Cleveland 20, Ohio.

Design—A five span deck girder bridge with cantilevers and suspended spans in second and fourth spans. Total length 555 ft. Two - 24 ft. roadways.

R. W. Ullman, 3057 Edgehill Road, Cleveland Heights 18, Ohio.
Design—A deck truss type 130 ft. in length. The 26 ft. roadway is supported by six trusses inclined to form three triangular space frames. Concrete slab acts integrally with truss top chords.

Henry F. Gauss, 721 East First Street, Moscow, Idaho.
Design—An overpass structure consisting of three simple deck spans carrying 24 ft. roadway and one sidewalk on four lines of girders.

Adam R. Werth, 1825 Summerfield Street, Brooklyn 27, New York.
Design—A three span continuous bridge using self-anchored arches of box shape for a total length of 1800 ft. Two - 24 ft. roadways.

R. Reikenis, 607 St. Paul Street, Baltimore 15, Maryland.
Design—A three span continuous deck girder bridge (skewed) having a total length of 299 ft. Two separate bridges, each with a 24 ft. roadway.

The remaining chapters are devoted to discussions and more elaborate descriptions of the above designs and a few of the other designs presented which did not win awards.

Summary of the Designs

This program did not restrict the length, width, or type of bridge. Consequently, the spans vary from the 32 ft. span for a simple beam of one entry to the 1800 ft. length for the three span continuous bridge of another entry. The minimum roadway width of any bridge presented is 26 ft. and, although a significant number of the designs utilize this width, the variation is remarkable. The greatest width is 86 ft. total which provides two - 36 ft. roadways, a median, and two sidewalks.

Of the different structural types submitted, the continuous beams and girders are the most numerous. Some have floorbeams and stringers to support the floor slabs, others support the slabs directly. A few of the continuous girders utilize suspended spans with cantilever arms in the intermediate spans. The principles of prestressing

and composite action with concrete slabs are included in some of these designs.

Simple beam spans, simple truss spans, continuous trusses, and arches are all represented in the designs presented. More participants selected deck bridges than through bridges. Reinforced concrete slabs constitute the roadways with but few exceptions. Standard steel shapes and sizes, except for a small number of bent plates, comprise the use of steel for all of the bridges.

For the longer or more complicated structures, the amount of effort required to prepare and enter an exhibit in this program was not small; nevertheless, the entries are consistently complete and contain an adequate amount of details. The program has been successful, considering both the number and quality of the designs. The authors represent 15 different states. Participation in this program was limited to citizens of the U.S.A.

The bridges are divided in the chapters that follow according to the classification of the principal structural member. Drawings showing the details of the designs are presented along with discussions and descriptions of the exhibits. The papers selected for presentation in this book are not given in their entirety—the amount included is sufficient, however, to bring out the more important material.

CHAPTER II

CONTINUOUS BEAMS AND GIRDERS

About forty per cent of the bridges entered in this program have some kind of a continuous beam or girder as the main structural member, however, there is a wide variation in span length, number of spans, and width of bridge. Also, the function of the continuous girders differs between designs. Because of the large number of this type of structural member and the differences in the manner in which it is employed, the bridges of this chapter are divided into three classifications.

Continuous Girder Bridges Without Floorbeams

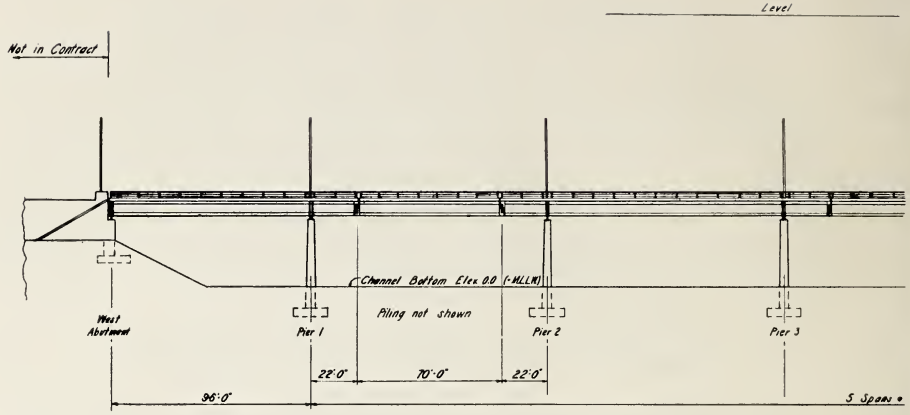
This classification includes haunched girders for the longer span bridges and constant depth girders for the shorter spans. One design has an expanded beam for the girder in the regions of low shear between the supports. Another has cantilever arms with suspended spans incorporated in a continuous girder bridge.

Elwyn H. King, Long Beach, California, designed a continuous girder bridge having a total length of 762 ft. for its seven spans. The end spans are 96 ft. long and the five center spans are 114 ft. in length. Spans 2, 4, and 6 each contain a suspended span of 70 ft. —leaving a cantilever arm of 22 ft. at both ends of each suspended span.

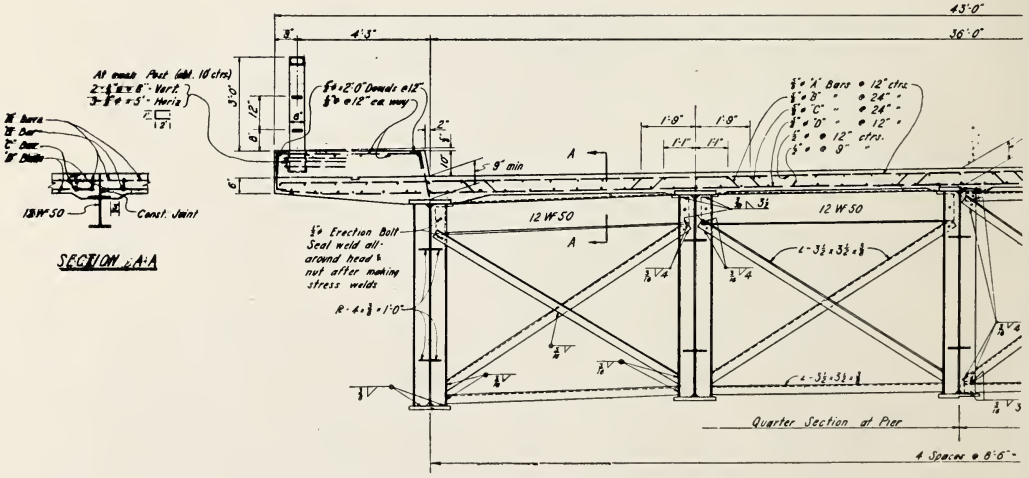
The bridge is 86 ft. wide with the deck divided at the center. Each half supported directly on five lines of girders has a portion of the median, a 36 ft. roadway, and 5 ft. of sidewalk slab. The reinforced concrete slab that spans the 8 ft.-6 in. between girders varies from 7 in. to 9 in. in thickness as shown in Figure 1. Figure 2 shows the spacing between cross frames varying from 22 ft. to 24 ft. Details of the welded girders are given in Figures 3 and 4 and details for some of the riveted girders in Figure 5. As can be seen the webs for all of the girders are 78 in. deep.

Mr. King commented as follows in discussing his designs:

"Both designs are for a bridge of seven spans totalling 762 ft. The bridge crosses a floodway, contained between levees, carrying storm waters to the sea during the rainy season. The bridge consists of multiple plate girder spans with alternate girders cantilevering beyond their supports to pick up suspended spans, thus affecting a considerable economy of material.


Level
Not in Contract
Channel Bottom Elev. 0.0 (-MLLW)
West Abutment
Pier 1
Piling not shown
Pier 2
Pier 3
22'-0"
70'-0"
22'-0"
96'-0"
5 Spaces

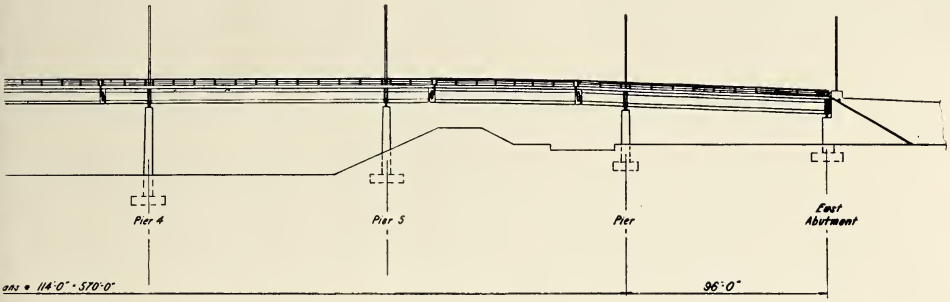
ELEVA


43'-0"
36'-0"
4'-3"
At ends Post (Add. 10 ctrs)
2-4"x6"x8'-0" Vert.
3-3/8"ø x 5'-0" Horiz
5/8"ø x 2'-0" Dowels @12" 5/8"ø @12" ea. way
3'-0"
12"
8"
2"
9" min
A
1'-0"
1'-9"
1'-1"
1'-1"
5/8"ø 'A' Bars @ 12" ctrs.
5/8"ø 'B' " @ 24" "
5/8"ø 'C' " @ 24" "
5/8"ø 'D' " @ 12" "
5/8"ø @ 12" ctrs.
5/8"ø @ 8" "
12 W 50
Const. Joint
SECTION on A-A
5/8" Eraction Bolt Seal weld all around head & nut after making stress welds
R-4x3/8 x 1'-0"
L-3 1/2 x 3 1/2 x 3/8
Quarter Section at Pier
4 Spaces @ 8'-6"

CROSS

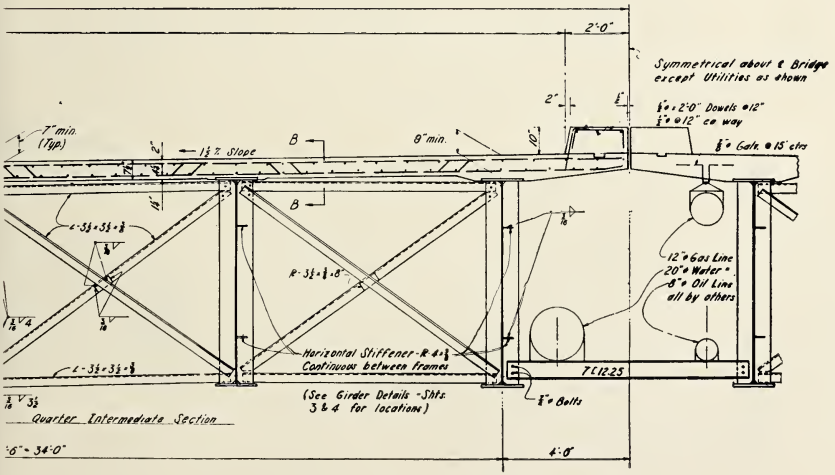
480 V.C. to 57

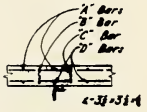
Not in Contract



VATION

For Framing Plan, see sheet 2
For Girder Details, see sheets 3 & 4





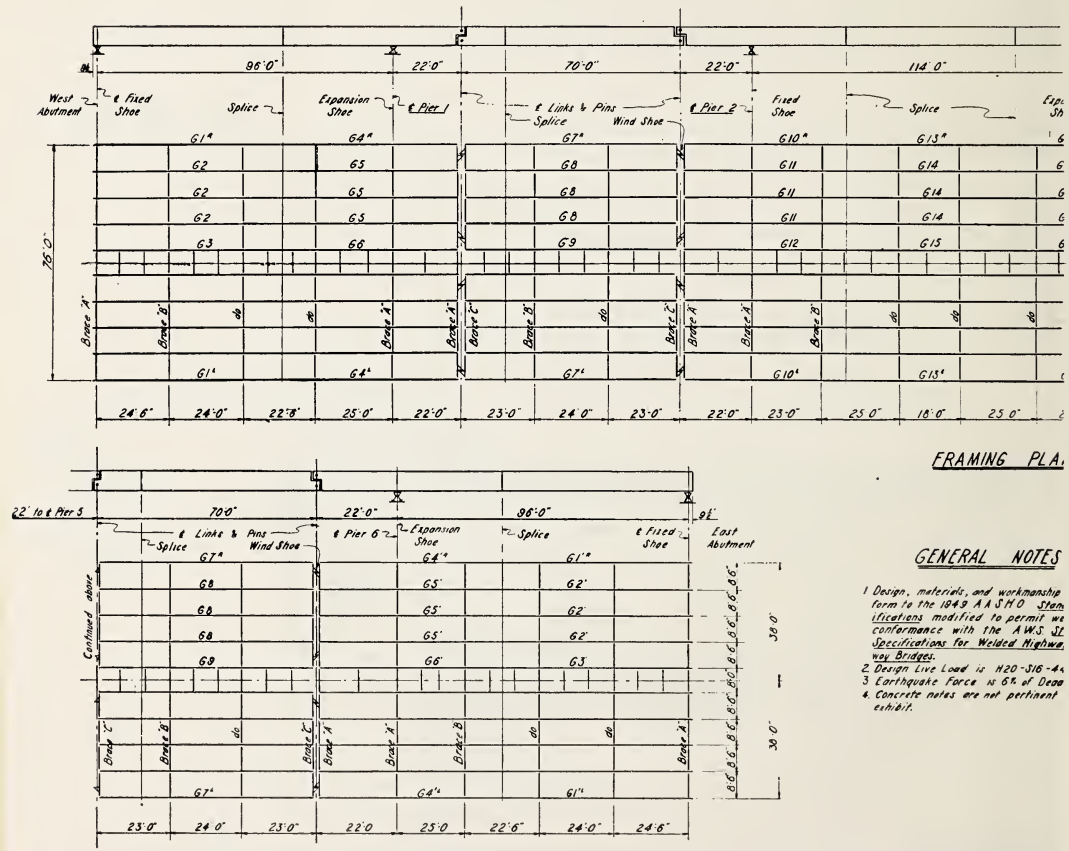
SECTION B-B

SS - SECTION

Figure 1

GIRDER REACTIONS						
	Abutments			Piers		
Mark	G1	G2	G3	G4 & 10	G5 & 11	G6 & 12
Dead Load	89.3	46.8	67.9	272.4	142.6	207.4
Live Load-Road	26.8	55.2	42.0	42.9	88.5	67.3
Impact	6.1	12.5	9.5	9.0	18.5	14.1
Live Load Sidewalk	14.4	—	—	34.2	—	—
TOTAL (kips)	136.6	114.5	119.4	338.5	249.6	288.8

										GIRDER STR
Mark		G1	G2		G3	G4	G5	G6	G7	
Girder Sections	Web ℔	78 × ⅜				78 × ⅜				
	Flange Pls or	16 × 1⅛ × 46	16 × 1 × 46		16 × 1 × 42	16 × 1⅛ × 20	16 × 1¼ × 20	16 × 1⅜ × 22	14 × 1 × 38	
	or	16 × ⅞	16 × ⅞		16 × ⅞	18 × 1⅛ × 20	16 × ⅞ × 18	16 × ⅞ × 22	14 × ⅞	
	or	—	—		—	18 × ⅞	18 × ⅞	18 × ¾	—	
Section Modulus - in³		1932	1620		1620	2545	1932	2162	1462	
Moments (Ft.-kips)		Max +	Max +	Max -	Max +	Max -	Max -	Max -	Max +	
	Dead Load	+ 1,675	+ 809	- 566	+ 1277	- 2,410	- 1,265	- 1,840	+ 1,460	
	Live Ld-Road	+ 584	+ 1,235	- 595	+ 931	- 562	- 1,160	- 885	+ 407	
	Impact	+ 134	+ 280	- 164	+ 211	- 161	- 330	- 250	+ 104	
	L Ld S'walk	+ 361	—	—	—	- 343	—	—	+ 209	
	Total	+ 2,704	+ 2,324	- 193	+ 2,419	- 3,476	- 2,755	- 2,975	+ 2,180	
	Reversal	—	+ 97	- 97	—	—	—	—	—	
	Design	+ 2,784	+ 2,421		+ 2,419	- 3,476	- 2,755	- 2,975	+ 2,180	
Section Modulus Reqd.		1856	1614		1612	2318	1837	1984	1454	
Shears (10)	Dead Load	89.3	46.8		67.9	136.2	71.3	103.7	83.5	
	Live Ld-Road	26.8	55.2		42.0	29.4	60.6	46.2	25.8	
	Impact	6.1	12.5		9.5	8.8	18.1	13.8	6.5	
	L Ld-S'walk	14.4	—		—	17.1	—	—	10.5	
	Total	136.6	114.5		119.4	191.5	150.0	163.7	126.3	

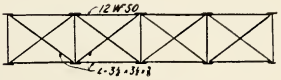


FRAMING PLA

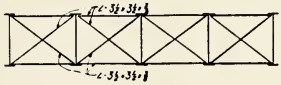
GENERAL NOTES

1. Design, materials, and workmanship form to the 1949 A A S H O Stan ifications modified to permit we conformance with the A.W.S. St Specifications for Welded Highwa way Bridges.
2. Design Live Load is H20-S16-44
3. Earthquake Force is 6% of Dead
4. Concrete notes are not pertinent exhibit.

STRESS TABLE										
7	G8	G9	G10	G11	G12	G13	G14		G15	
	78 × ⅞			78 × ⅞			78 × ⅞			
38	14 × ⅞ × 30	14 × ⅞ × 34	16 × 1⅞ × 24	16 × 1⅞ × 18	16 × 1⅞ × 22	16 × 1⅝ × 30	16 × 1⅝ × 58		16 × 1⅝ × 50	
⅝	14 × ⅝	14 × ⅝	16 × 1⅞ × 20	16 × 1⅞ × 18	16 × 1⅞ × 20	—	—		—	
—	—	—	16 × ⅞	16 × ⅞	16 × ⅞	—	—		—	
52	1262	1323	2545	1932	2162	2004	2004		1774	
↕	Max +	Max +	Max −	Max −	Max −	Max +	Max +	Max −	Max +	Max −
160	+ 765	+ 1,114	- 2,410	- 1,265	- 1,840	+ 1,460	+ 765	- 535	+ 1,112	- 778
107	+ 840	+ 639	- 562	- 1,160	- 885	+ 733	+ 1,510	- 1,160	+ 1,150	- 882
04	+ 213	+ 163	- 161	- 330	- 250	+ 152	+ 315	- 240	+ 240	- 183
09	—	—	- 343	—	—	+ 550	—	—	—	—
80	+ 1,818	+ 1,916	- 3,476	- 2,755	- 2,975	+ 2,895	+ 2,590	- 865	2,302	- 287
—	—	—	—	—	—	—	432	432	143	143
90	+ 1,818	+ 1,916	- 3,476	- 2,755	2,975	+ 2,895	3,002		2,645	
54	1212	1276	2318	1837	1984	1,930	2014		1764	
15	438	637	1362	713	1037	643	335		612	
18	53.1	404	294	606	462	198	43.8		33.9	
5	135	103	88	181	138	42	9.2		7.2	
5	—	—	171	—	—	92	—		—	
3	1104	1144	1913	1500	1637	971.5	86.2		1023	



BRACE A



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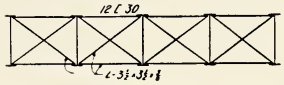

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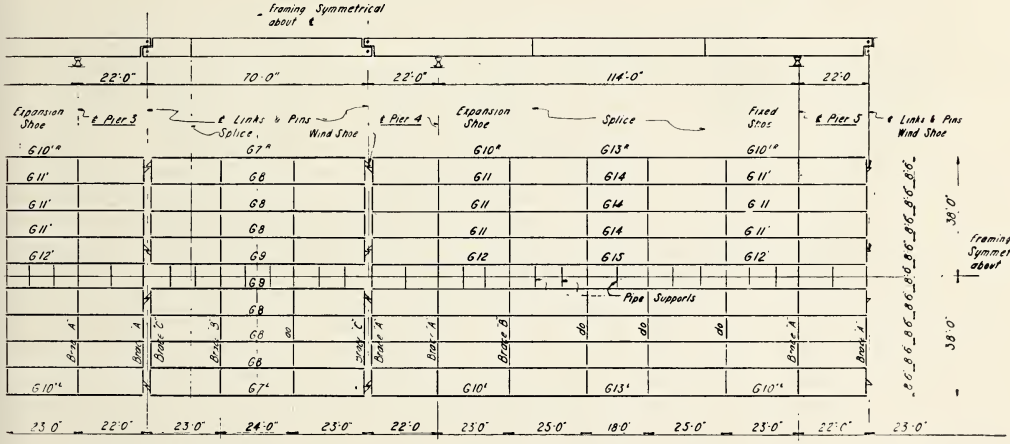


PLAN

TES

nship shall con-
Standard Spec-
it welding in
S Standard
ghway & Rail"
'6"-44.
Dead Load.
nent to this

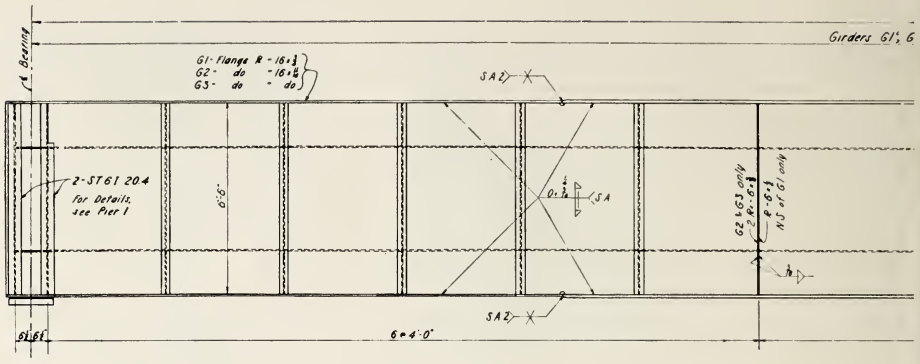
STRUCTURAL STEEL NOTES

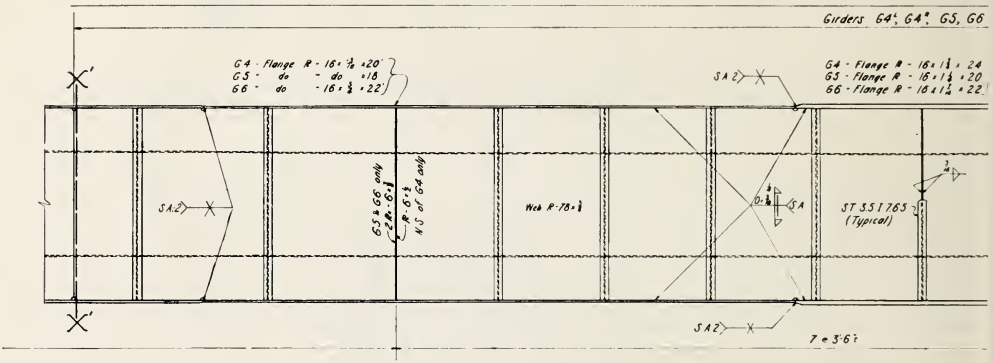
1 All material shall be Structural Carbon Steel conforming to A.S.T.M.-A7-46
2 Flange to over 1½" & Flange to between 1" & 1½" whose carbon content plus one-sixth the manganese content exceeds 0.40% shall be marked according to Article 504(7) of the welding specifications.
3 All spans shall be cambered for full dead load plus one-half live load
4 Vertical stiffeners shall be normal to the flanges.
5 No field splices are provided since a similar bridge across upstream was erected without splices.

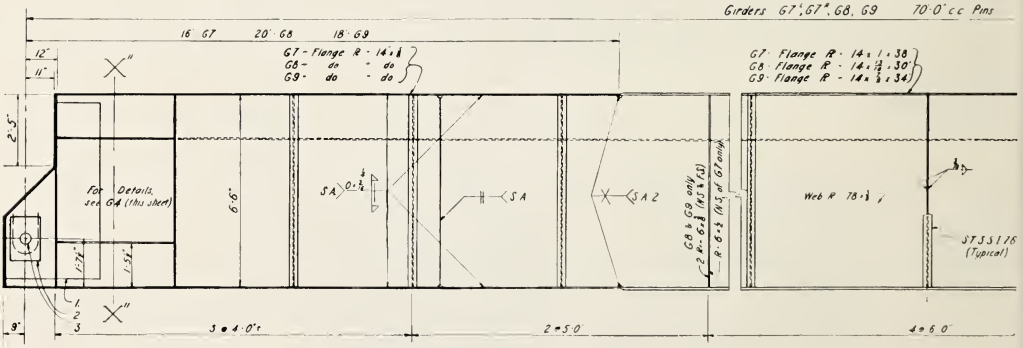
WELDING NOTES

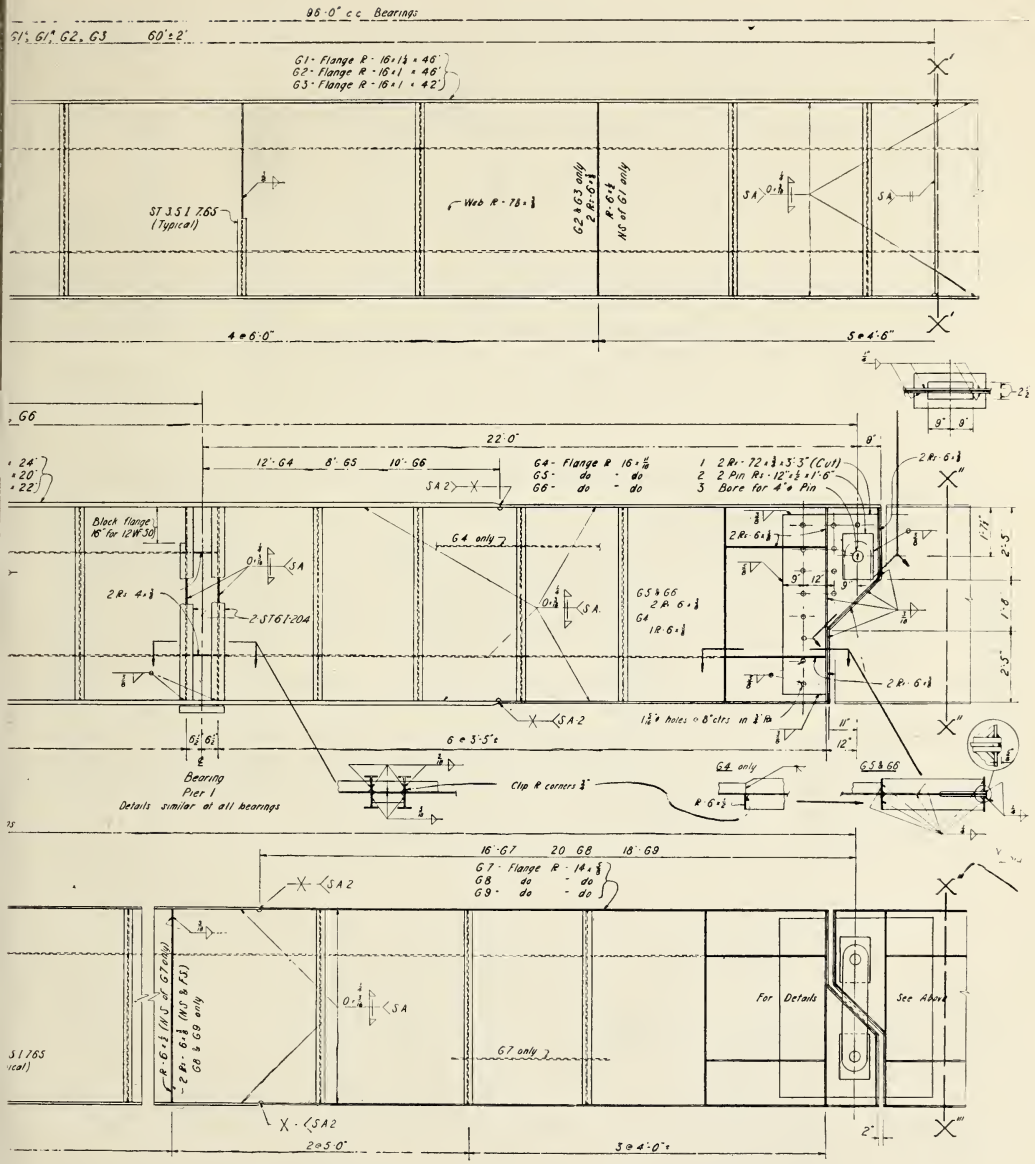
1 S.A.-1 All welds marked "S.A." shall be full penetration welds made by the automatic submerged arc process, using one pass on each side of the joint.
2 S.A.-2 All welds marked "S.A.2" shall conform to "1 above. Joint details shall be as shown in the table, "Flange Butt Welds," on sheet "4
3 Horizontal and vertical non-bearing stiffeners may be welded by either shielded arc or submerged arc processes.
4 Joints having partial length welds shall also have a full length seal weld.
5 Unless specifically shown otherwise, all welds on the far side of the web shall be the same as shown on the near side.

Figure 2


Girders G1, G6
℄ Bearing
G1 - Flange R - 16 x 3/4
G2 - do - 16 x 7/8
G3 - do - do
SA 2
2 - ST 61 20.4
For Details, see Pier 1
6'-6"
SA
G2 & G3 only
2 - R - 6 x 1/2
N.S. of G1 only
6 @ 4'-0"


Girders G4, G4*, G5, G6
G4 - Flange R - 16 x 1 1/8 x 20
G5 - do - do x 18
G6 - do - 16 x 1 3/8 x 22
G4 - Flange R - 16 x 1 1/8 x 24
G5 - Flange R - 16 x 1 1/8 x 20
G6 - Flange R - 16 x 1 1/4 x 22
SA 2
G5 & G6 only
2 - R - 6 x 1/2
N.S. of G4 only
Web R - 78 x 3/8
SA
ST 35 1765
(Typical)
7 @ 3'-6"

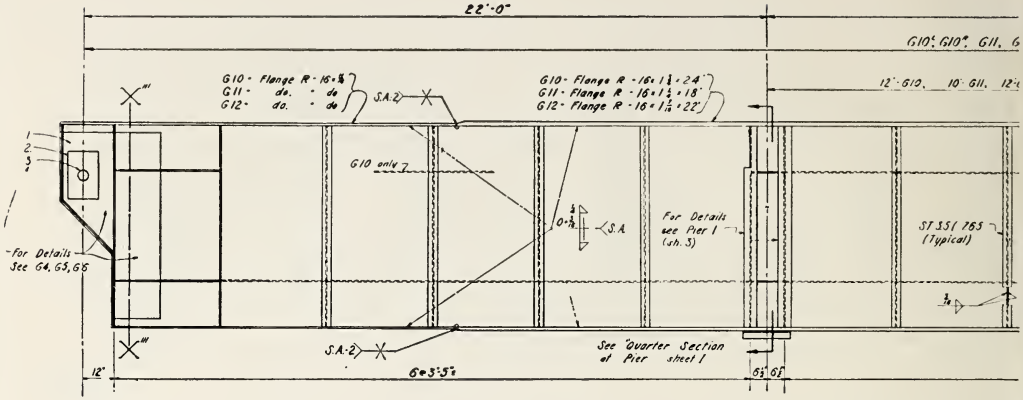

Girders G7, G7*, G8, G9 70'-0" c.c. Pins
16' G7 20' G8 18' G9
G7 - Flange R - 14 x 7/8
G8 - do - do
G9 - do - do
G7 - Flange R - 14 x 1 x 38
G8 - Flange R - 14 x 1 1/2 x 30
G9 - Flange R - 14 x 1 1/2 x 34
12"
11"
2'-5"
For Details, see G4 (this sheet)
6'-6"
SA
SA 2
G8 & G9 only
2 - R - 6 x 1/2 (N.S. of G7 only)
Web R 78 x 3/8
ST 35 178
(Typical)
1'-7 1/2"
1'-5"
9"
3 @ 4'-0"
2 @ 5'-0"
4 @ 6'-0"

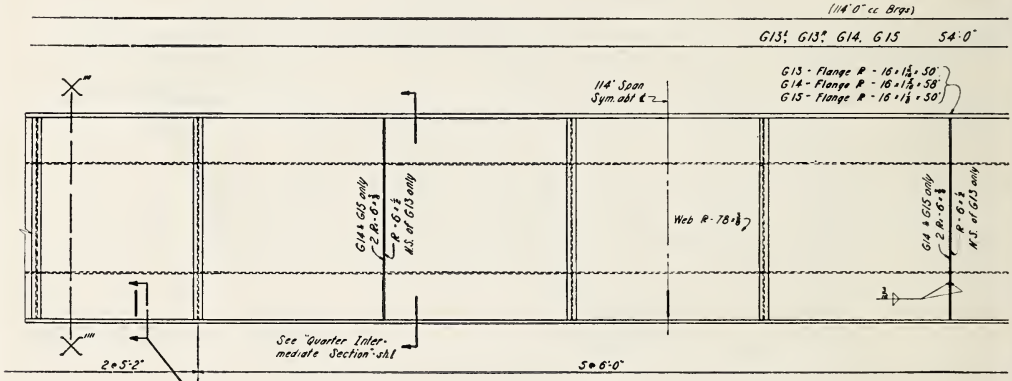

98'-0" c.c. Bearings
G1, G1*, G2, G3 60'±2"
G1 - Flange R - 16×1½ × 46"
G2 - Flange R - 16×1 × 46"
G3 - Flange R - 16×1 × 42"
ST 351 765 (Typical)
Web R - 78×⅜
G2 & G3 only 2 Rs - 6×⅜
R - 6×⅜ N.S. of G1 only
SA
4 @ 6'-0"
5 @ 4'-6"
G6
22'-0"
12'-G4 8'-G5 10'-G6
G4 - Flange R 16×11/16
G5 - do - do
G6 - do - do
1 2 Rs - 72×⅜×3'-3" (Cut)
2 2 Pin Rs - 12"×⅜×1'-6"
3 Bore for 4"ø Pin
Block Flange 16" for 12WF30
G4 only
2 Rs 4×⅜
2 ST 61-20A
G5 & G6 2 R 6×⅜
G4 1 R 6×⅜
2 Rs 6×⅜
1⅛" holes @ 8" ctrs in ⅜" fls
6 @ 3'-5" c
Bearing Pier 1
Details similar at all bearings
Clip R corners ⅜"
R 6×⅜
16'-G7 20 G8 18'-G9
G7 - Flange R - 14×⅝
G8 do - do
G9 - do - do
R 6×⅜ (N.S. of G7 only)
2 Rs - 6×⅜ (N.S. & S.S.) G8 & G9 only
G7 only
For Details
See Above
2 @ 5'-0"
3 @ 4'-0"

NOTES

G1*, G4*, G7*, G10* & G13* are as shown G1', G4', G7', G10', & G13' have vertical stiffeners on far side and horizontal stiffeners on near side (exterior face). G1, G2, G3, G4, G5', & G6' same as G1, G2 G3, G4, G5 & G6, except make opposite hand

Figure 3


22'-0"
G10, G10, G11, G
G10 - Flange R - 16 x 7/8
G11 - do. - do
G12 - do. - do
S.A.2
G10 - Flange R - 16 x 1 1/2 x 24"
G11 - Flange R - 16 x 1 1/2 x 18"
G12 - Flange R - 16 x 1 1/16 x 22"
12'-G10, 10'-G11, 12'-
G10 only
For Details See G4, G5, G6
S.A.
For Details see Pier I (sh. 3)
ST 3.5 I 7.65 (Typical)
S.A.2
See "Quarter Section at Pier" sheet 1
12"
6@3'-5"
6 1/2"
6'-5"


(114'-0" c.c. Brgs)
G13, G13, G14, G15
54'-0"
G13 - Flange R - 16 x 1 1/2 x 50"
G14 - Flange R - 16 x 1 1/2 x 58"
G15 - Flange R - 16 x 1 1/2 x 50"
114" Span
Sym. about ℄
G14 & G15 only
2 R - 6' - 5/8
R - 6' - 5/8
N.S. of G13 only
Web R - 78 x 3/8
G14 & G15 only
2 R - 6' - 5/8
R - 6' - 5/8
N.S. of G13 only
See "Quarter Intermediate Section" sht
2@5'-2"
5@6'-0"

Median Girder Only (Typical)

FLANGE BUTT WELDS

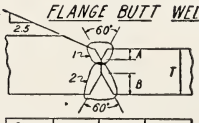
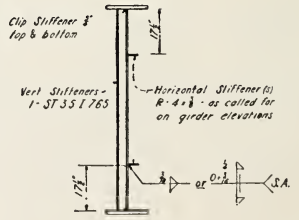
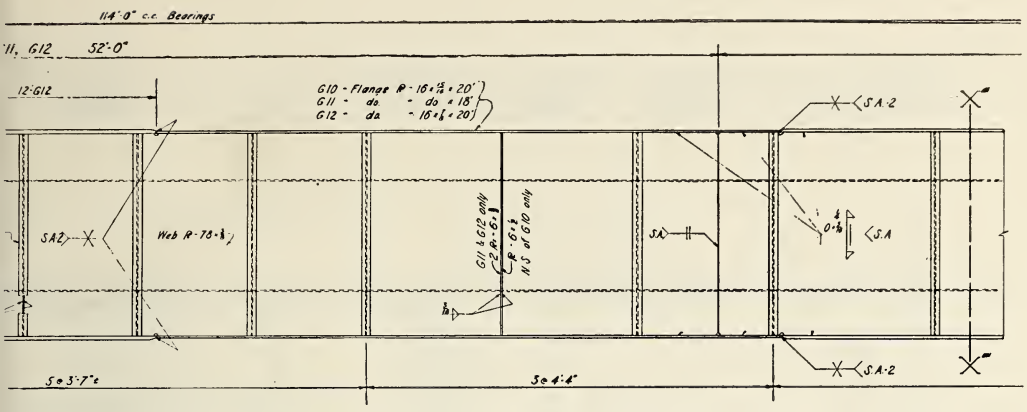
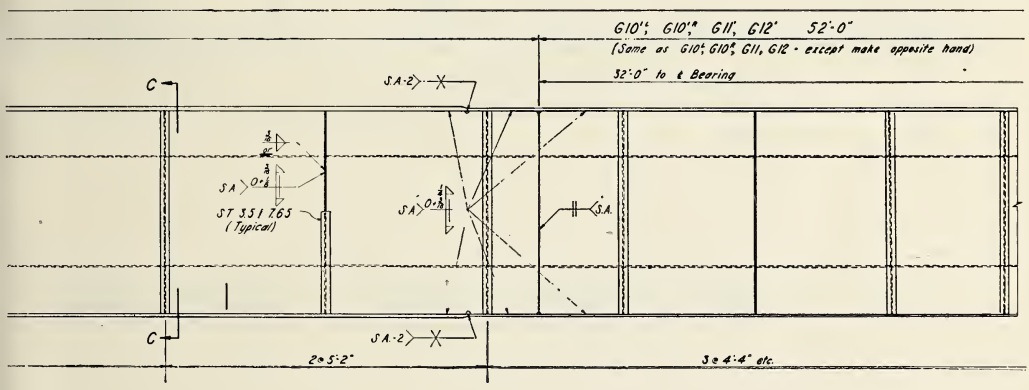


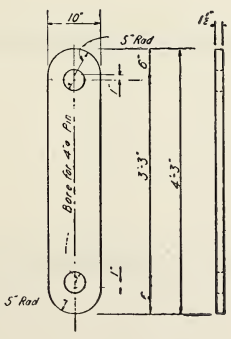
Plate Thickness T	Pass	A	B
5/8, 11/16	1	0	
	2		0
3/4, 13/16	1	1/8	
	2		1/8
7/8, 15/16	1	1/4	
	2		1/4


Clip Stiffener 3/4" top & bottom
1'-7 1/2"
Vert Stiffeners - 1 - ST 3.5 I 7.65
Horizontal Stiffener(s) R - 4 x 3/8 - as called for on girder elevations
1'-7 1/2"
3/8
or
S.A.

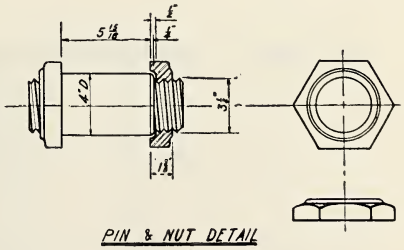
SECTION C-C





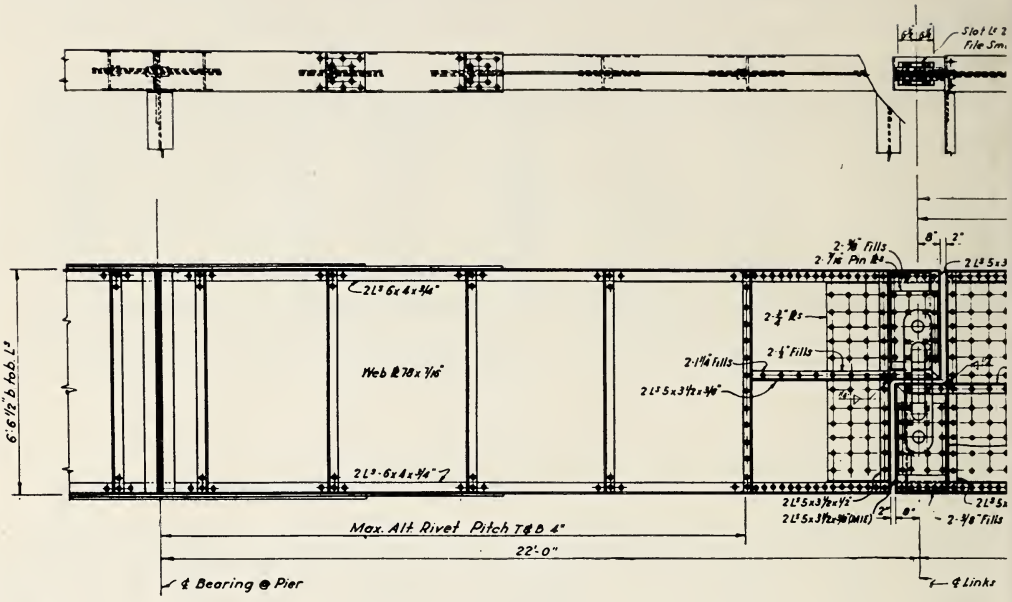


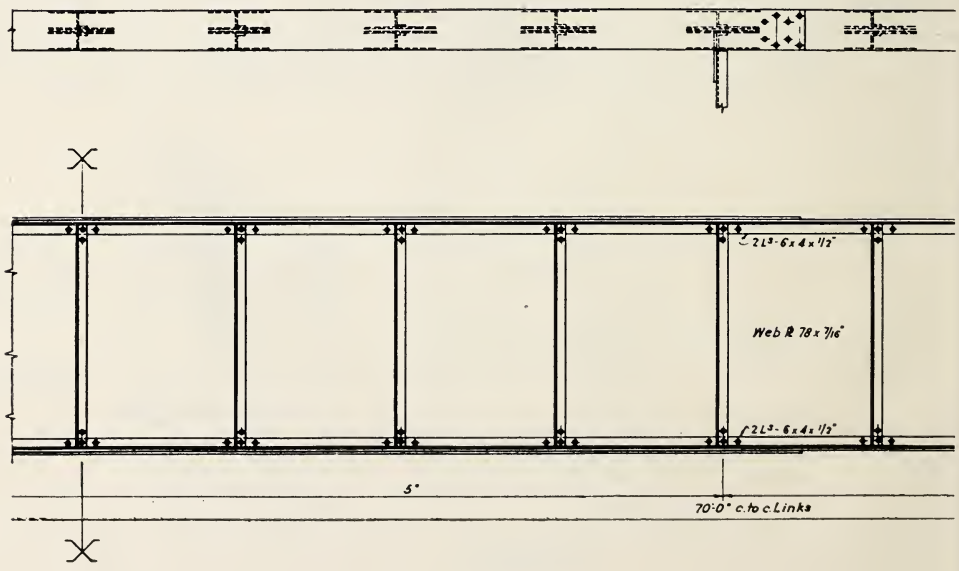
LINK DETAIL
(120 Reqd)

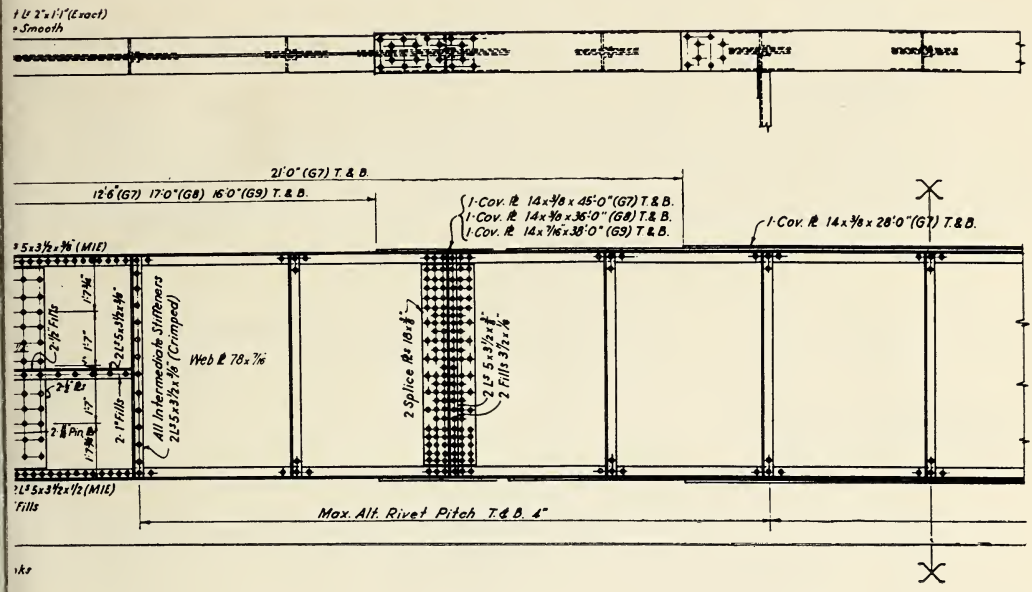


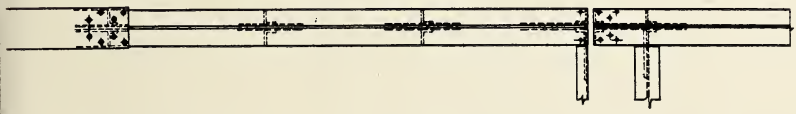
PIN & NUT DETAIL

Figure 4


2 L³ 6 x 4 x ¾"
Web PL 78 x 7/16"
2 L³ 6 x 4 x ¾"
2 L³ 5 x 3½ x ⅜"
2-1¾" Fills
2-⅞" Fills
2-⅞" PLs
2-⅞" Fills
2 L³ 5 x 3½ x ½"
2 L³ 5 x 3½ x ½" (Mill)
2-⅞" Fills
Max. Alt Rivet Pitch T & B 4"
22'-0"
℄ Bearing @ Pier
℄ Links


2 L³ - 6 x 4 x 1/2"
Web PL 78 x 7/16"
2 L³ - 6 x 4 x 1/2"
5"
70'-0" c. to c. Links


21'0" (G7) T. & B.
12'6" (G7) 17'0" (G8) 16'0" (G9) T. & B.
1-Cov. Pl 14x⅜x45'0" (G7) T. & B.
1-Cov. Pl 14x⅜x36'0" (G8) T. & B.
1-Cov. Pl 14x7/16x38'0" (G9) T. & B.
1-Cov. Pl 14x⅜x28'0" (G7) T. & B.
Web Pl 78x7/16
All Intermediate Stiffeners
2 Splice Pls 18x⅜
Max. Alt. Rivet Pitch T. & B. 4"



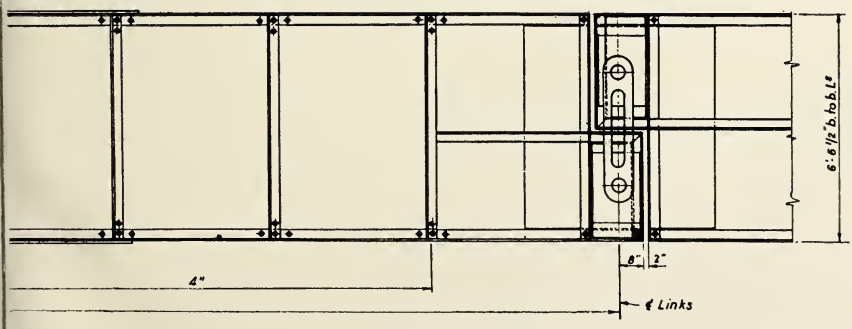
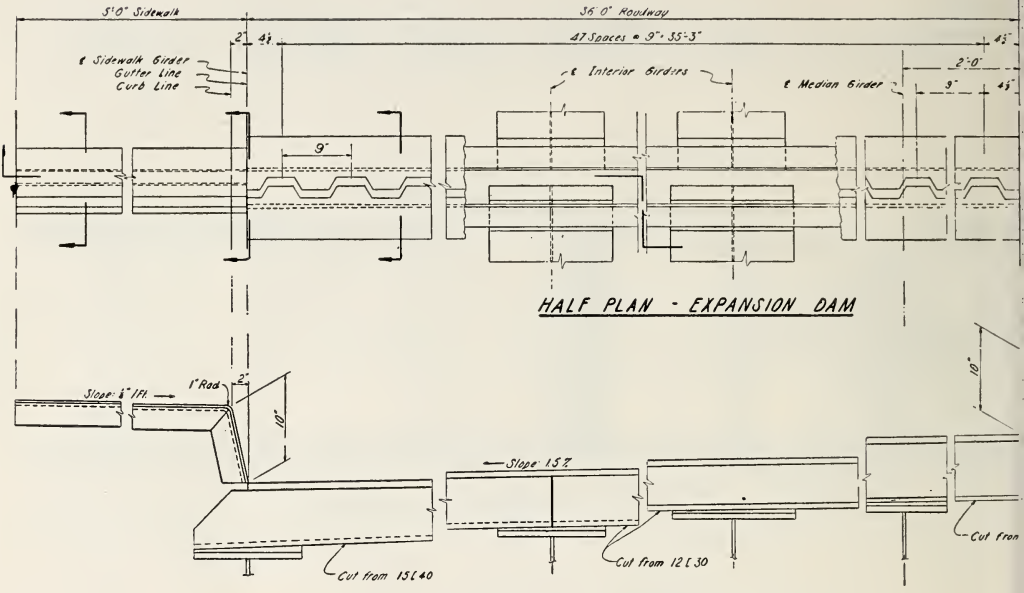
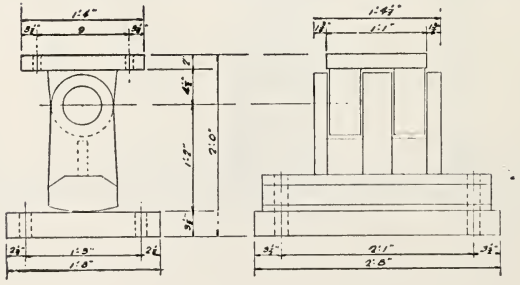
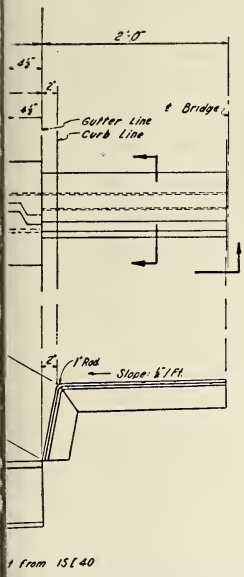
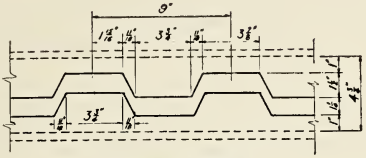

6'6 1/2" b. to b. Ls
8"
2"
4"
℄ Links

Figure 5

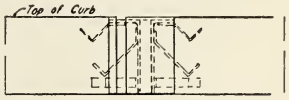

5'-0" Sidewalk
36'-0" Roadway
47 Spaces @ 9" = 35'-3"
℄ Sidewalk Girder
Gutter Line
Curb Line
℄ Interior Girders
℄ Median Girder
9"
2'-0"
HALF PLAN - EXPANSION DAM
Slope: ½" I.Ft.
1" Rad.
10"
Slope: 1.5%
Cut from 15[40
Cut from 12[30
SECTION D-D


1'-6"
9"
1'-4½"
1'-1"
2'-0"
1'-3"
1'-8"
2'-1"
2'-8"

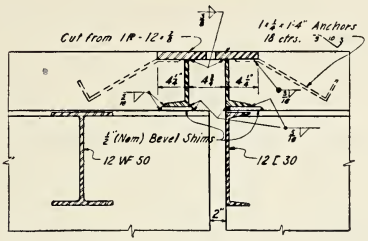




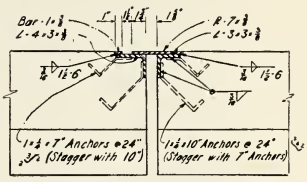
DETAIL OF TEETH



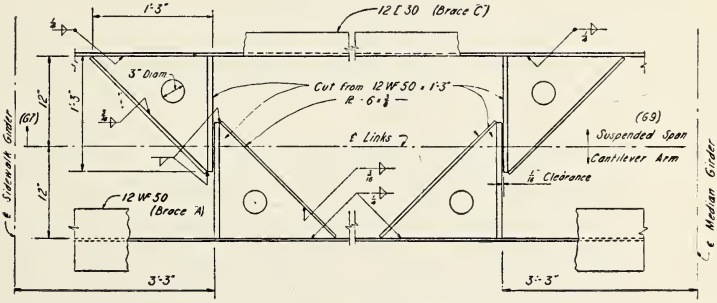
SECTION F-F



SECTION E-E



SECTION G-G



WIND SHOES

Figure 6

"Exhibit B has further limitations than imposed by the contest rules. The designer, desiring the most reliable comparison between welded and riveted construction, imposed on himself the limitation of accepting the deck, girder spacing, and shoes of Exhibit A. To that extent, Exhibit B is not original. However, everything else was planned by the designer, from the moment diagrams to the welding details.

"The outstanding advantage of Exhibit B is its marked saving of structural steel, the objective of the contest. The 326 tons saved amount to 23.7 per cent of the steel used in Exhibit A, which, incidentally, has been erected. It is estimated the financial saving would be about $103,000.

"Exhibit B possesses no other advantages over Exhibit A. Erection methods would be identical (Exhibit A was erected without field splicing.) Maintenance would be approximately the same.

"It is believed Exhibit B has two distinct features of sufficient merit to enumerate them. The major item is the use of automatic submerged arc welding in preference to shielded arc. The former possesses the following advantages over the latter:

1. Better quality weld (more uniform)
2. Less shrinkage (unless an uneconomical method of manual welding was used)
3. Less distortion
4. More predictable shrinkage and distortion
5. (As a result of #4) Easier correction for shrinkage and distortion

"The above advantages fully justify the specifying of the automatic submerged arc welding process.

"The second advantage lies in the use of horizontal stiffeners on the girders. From an engineering viewpoint, the use of these horizontal stiffeners saved steel (and money) by reducing the required web thickness one-sixteenth of an inch. From the public's viewpoint, a more esthetic structure resulted with clean, horizontal, functional lines replacing the cut up lines of girders with vertical stiffeners only."

The riveted structure weighs 2,753,100 lb. and cost $460,000 (bid price was $0.167). The welded bridge weighs 2,100,200 lb. and the estimated price of $0.17 would result in a cost of $357,000. The shop welds amount to 16,688 lb. and the field welds to 560 lb. Details of the shoes and expansion dam are shown in Figure 6.

Kiser E. Dumbauld, Columbus, Ohio, presented a bridge of eight continuous girder spans. At the longitudinal centerline of the

bridge, the overall length is 798 ft., consisting of 78 ft. end spans and six center spans of 107 ft. each. Because half of the bridge is skewed, the girders in the two spans adjacent to the center pier in the skewed half are unequal in length, being longer than 107 ft. on one side and shorter on the other side. The total width of the bridge is 70 ft. which includes a 2 in. open joint that divides the bridge into two parts. There is a sidewalk and a 26 ft. roadway on both sides of the split median. Figure 7 shows the general plan and elevation and Figure 8 shows the framing plan for structural steel.

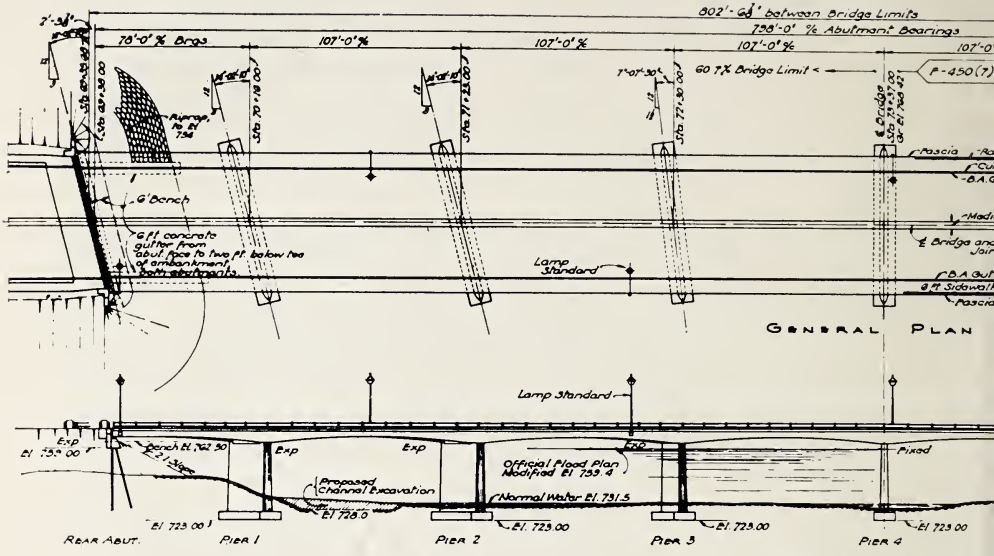
The 7½ in. reinforced concrete slab is supported directly on the girders which are spaced 8 ft. apart. The slab is covered with a 2½ in. surface coat of asphaltic concrete as shown in Figure 9. The sidewalk is also reinforced concrete and is 7 ft. wide from the curb to the outside edge. Mr. Dumbauld stated that the use of concrete for the sidewalk, curb, and fascia required ten lines of girders instead of the eight lines which could have been used had these items been of steel—supported by light steel brackets attached to the outside girders.

The parabolic curves for the bottom flanges of the twelve different lengths of girders are incorporated into one diagram on Figure 8. These haunched girders vary in depth from 3 ft. at the ends and midway between piers to 6 ft. at the piers. Figure 10 indicates the changes in web thickness from 7/16 in. to 3/8 in. to 5/16 in. The flange plates are 16 in. wide for the inside girders. The fascia girders have 14 in. flanges.

In discussing his Exhibit A2, Mr. Dumbauld stated: "The estimated traffic count for this structure is 13,500 vehicles per day by year 1970, with a predicted frequency per lane of heavy loaded combination vehicles of 400 every 24 hours. This structure should be placed under contract in summer of 1952 if material is available.

"This bridge was designed to replace a weak, narrow bridge nearby and improve the roadway intersection with a new highway located to bypass the city. This industrial city has a population of 12,000. The drainage area is a round shaped basin of 4,850 square miles. This bridge is located over the source of a river formed by the junction of two other streams. The abutments and piers were located to fit the banks and stream flow of the two tributaries, which resulted in the planning of a structure skewed at one end and square at the other. . . .

* * *



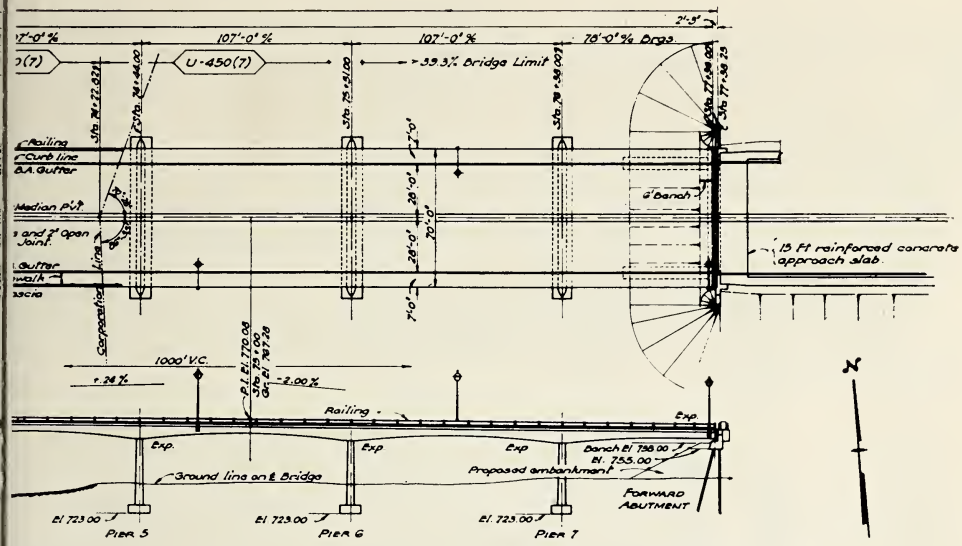
GENERAL PLAN

GENERAL ELEVATI
PIER - PILING NOT SHOW

ESTIMATED QUANTITIES

ITEM	TOTAL	UNIT	DESCRIPTION	FEDERAL AID PROJECT No F-450(7) [60.7% Deck]									FEDERA	
				REAR ABUT.	PIERS 1	2	3	4	SUPER-STRUCTURE	GEN-ERAL	TOTAL FOR F-450(7)		PIE 5	G
E-2	*Lump	Sum	Cofferdams, cribs and sheeting							Lump	Lump			
E-2	3,410	Cu yd	Unclassified excavation	80	580	280	350	250			1520		570	58
E-3	9723	Cu yd	Channel excavation							9723	9723			
S-1	1,560	Cu yd	Class 'C' concrete, superstructure						947		947			
S-1	173	Cu yd	Class 'E' concrete, abutments	90							90			
S-1	2,213	Cu yd	Class 'E' concrete, pier walls		333	334	326	317			1310		311	30
S-1	870	Cu yd	Class 'E' concrete, pier footings		129	129	126	126			510		120	12
S-3	4,700	Sq yd	Type 'C' waterproofing						2850		2850			
S-4	460,820	lb	Reinforcing steel	5840	7080	7800	7740	7660	245070	150	282090		7550	749
S-7	1863600	lb.	Structural steel						1131200		1131200			
S-8	1863600	lb	Field painting of structural steel						1131200		1131200			
S-14	1,598	Lin ft	Railing (steel)						970		970			
S-16	*Lump	Sum	First test pile							Lump	Lump			
S-17	*Lump	Sum	Pile test load							Lump	Lump			
S-17	*Lump	Sum	Subsequent test load							Lump	Lump			
S-18	16,890	Lin ft	12" Cast-in-place reinforced concrete piling	500	2160	2230	2230	2070			9190		2390	259
S-24	*Lump	Sum	Removal of existing structure							Lump	Lump			
S-25	*Lump	Sum	Bridge lighting system (standards, pull boxes)							Lump	Lump			
S-25	1,600	Lin ft	Electric conduit (3" fiber conduit including fittings)							970	970			
S-25	42	Lin ft	Electric conduit (1¼" flexible conduit including fittings)							24	24			
S-29	1,594	Lin ft	Subdrainage for wearing surface course						968		968			
T-10	175	Sq yd	Type 'A' riprap							175	175			
T-14	200	Lin ft	Concrete gutter	102							102			
T-35	321	Cu yd	Asphaltic concrete surface course, Type 'A' or 'C' (To AD or AT-100)						195		195			

* Lump Sum Items to be subdivided as follows -
60.7 % apportioned to Fed. Aid Project No. F-450(7);
39.3 % apportioned to Fed. Aid Project No. U-450(7).



ATION
HOWN

ERAL AID PROJECT NO. U-450(7) [39.3% Dock]

PIERS G	PIERS 7	FORMS ABUT	SUPER-STRUCTURE	GEN-ERAL	TOTAL FOR U-450(7)		
				Lump	Lump		
580	670	70			1890		
			G13		G13		
		86			86		
302	290				909		
120	120				360		
			1850		1850		
7490	7200	5756	156640	94	186730		
			732400		732400		
			732400		732400		
			G28		G28		
				Lump	Lump		
				Lump	Lump		
				Lump	Lump		
2590	2390	530			7700		
				Lump	Lump		
				Lump	Lump		
				G30	G30		
				18	18		
			G26		G26		
		98			98		
			126		126		

General Notes

REFERENCE shall be made to Standard Drawing No. RB-1-47 revised 7-27-49

REMOVAL OF EXISTING BRIDGE CO-36-705:- Existing structure shall be removed. Structural steel and strip floor shall be carefully dismantled and piled along the right-of-way for removal by the County forces. (The County Forces will match mark the structural steel.) The remainder of the structure shall become the property of the Contractor.

The West abutment shall be removed to Elev. 757.0 and East abutment to 6' below existing ground line. Banks in back of abutments shall be dressed to 2:1 slope.

Suitable waste masonry may be used as bank protection at the direction of the Engineer.

PILING shall be driven to a minimum bearing capacity of 45 tons for piers and 35 tons for abutments.

EXCAVATION quantity includes the removal of fill material between top of earth bench and bottom of abutment.

WELDING shall be Class 'A' except as shown. Any welds shown as field welds, may, at the option of the Contractor, be made in the shop.

REINFORCING STEEL splice lengths shall be not less than 30 diameters of the smaller bar involved.

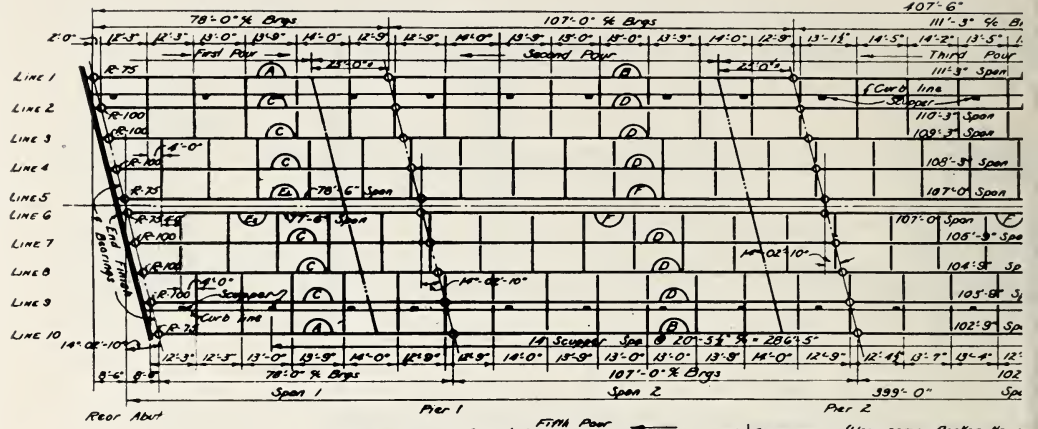
SURFACE FINISH:- Abutment wings, concrete end posts, faces of curbs and fascias of deck shall receive a rubbed surface finish. All other exposed surfaces shall be governed by the provisions of Item 5-1.

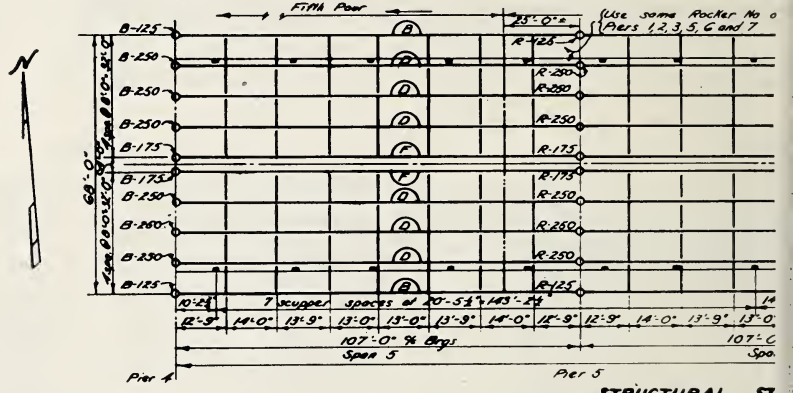
CONCRETE GUTTER 3 shall be 6'-0" wide and 6 inches thick, and shall be depressed 6 inches at the center. They shall extend from face of abutment down to 2'-0" below top of embankment and shall be centered under scuppers. Reinforcing bars ½"ø at 1'-6" centers, both directions, included in price per lineal foot.

TEST LOADS ON PILES shall be applied where directed by the Engineer. The magnitude of each test load shall be equal to 3 R unless the yield point is reached at a lesser tonnage.

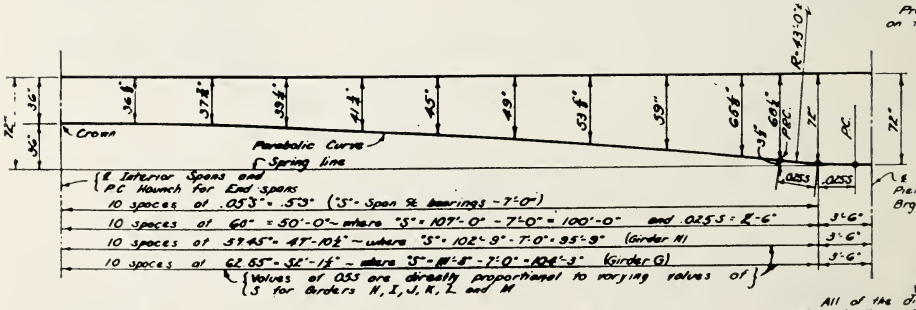
UTILITIES:- The Contractor shall cooperate with the utility companies in their installation and protection of any utility lines on the project.

Figure 7





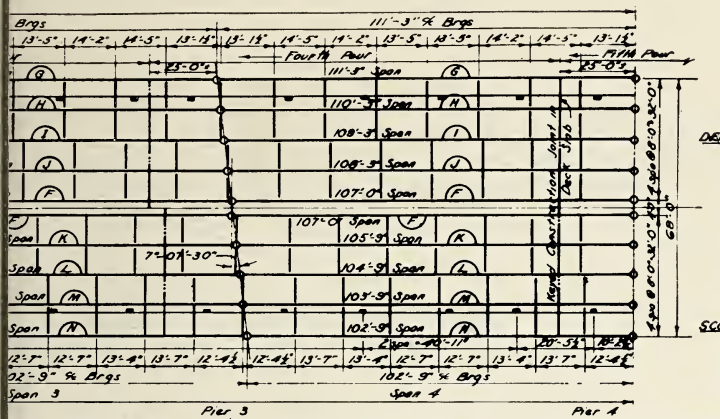
STRUCTURAL ST


Crown
Parabolic Curve
Spring line
℄ Interior Spans and PC Haunch for End Spans
10 spaces at .053" = .53" ("S" = Span ℄ bearings - 7'-0")
10 spaces at 60" = 50'-0" - where "S" = 107'-0" - 7'-0" = 100'-0" and .025 S = 2'-6"
10 spaces at 57.45" = 47'-10½" - where "S" = 102'-9" - 7'-0" = 95'-9" (Girder N)
10 spaces at 62.65" = 52'-1½" - where "S" = 111'-3" - 7'-0" = 104'-3" (Girder G)
Values of .053 are directly proportional to varying values of S for Girders N, I, J, K, L and M
℄ Pier Brg

TEMPLATE FOR PARABOLIC HAUNCH FOR WEB PLATES

WELDING

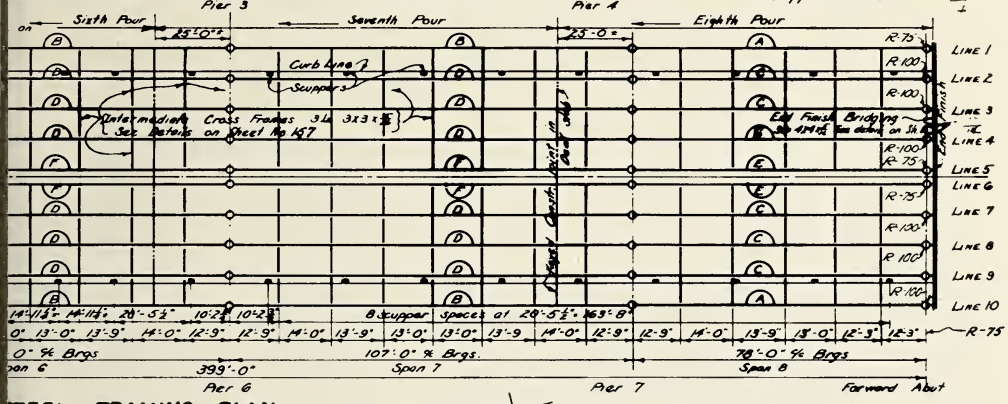
All of the diagonal
welded to the outstanding
In spans 1, 2, 3 and
welded to the outstanding
lines 1, 3, 5, 6, 8 and 10, the
do not occur at the line o
and 10 in spans 1, 2, 3 and
stiffeners provided the
horizontal frame angles
angles and welded to the



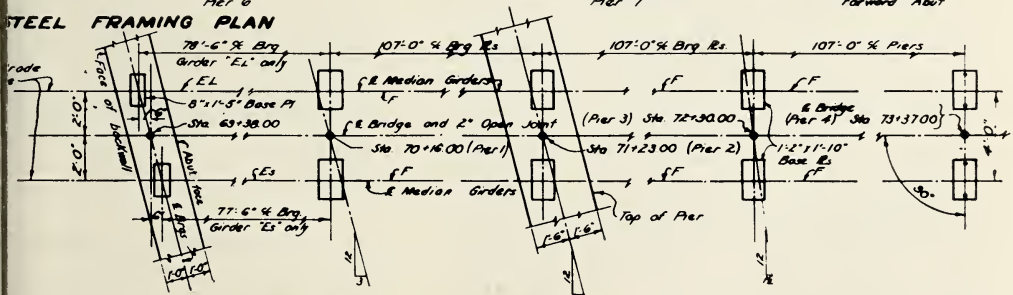
DECK SLAB CONSTRUCTION PROCEDURE. The deck slab concrete shall be placed in sections, between transverse keyed construction joints, in the numerical order and in the direction indicated in the structural steel framing plan, in order that the major portion of dead load deflection may occur prior to placing concrete over each pier.

This procedure may start from either end of the bridge or on either side of the longitudinal center line.

SCUPPERS. Nominal spacing of scuppers to be 20'-5½" ℄ to ℄ at 20 ft lengths of B.A. gutters. Change spacing slightly where necessary to miss piers and cross frame angles. See DA. No 160 for scupper details.



STEEL FRAMING PLAN



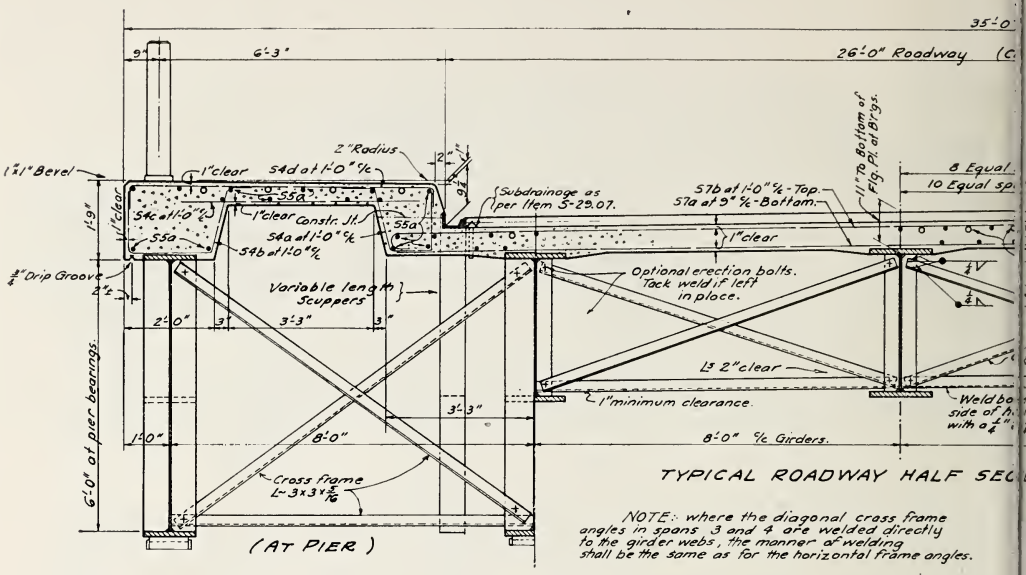
PART FRAMING PLAN FOR MEDIAN GIRDER BEARINGS ON SKEWED SPANS

...T AND FITTING OF FRAME ANGLES

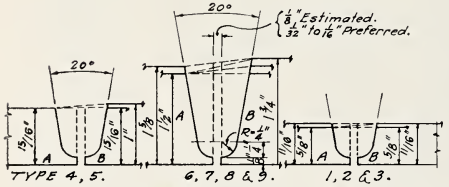
...cross frame angles over Piers 4, 5, 6 and 7, and in spans 5-6, 7 and 8 shall be ... legs of the stiffeners.

...4 and over piers 1, 2, and 3, the diagonal cross frame angles shall be ...g legs of the stiffeners on lines 2, 4, 7 and 9 of the roadway girders. On girder ...he diagonal cross frame angles may be welded to the girder flanges at the stiffeners ...or cross frames. However, the intermediate stiffeners on girders, in lines 1, 3, 5, 6, 8 ...4 may be shifted to provide a welded connection of diagonal cross frames to ...clear space between stiffeners does not exceed the unsupported depth of web. All ...shall be located a minimum of 1" clear of bottom flange and 2" clear of the diagonal ...web plates.

Figure 8



NOTE: where the diagonal cross frame angles in spans 3 and 4 are welded directly to the girder webs, the manner of welding shall be the same as for the horizontal frame angles.



FULL SIZE CROSS SECTIONS OF BUTT WELDS.

FIELD WELDING ~ "U" JOINTS ~

TYPE	PL. SIZE A	PL. SIZE B	NET AREA SQ. IN.	NET WT. lb. p.l.f.	Add for Reinf	GROSS WEIGHT NET+50%	LENGTH OF JT. inches	No. of JOINTS	TOTAL LENGTH IN FT.	TOTAL GROSS WT. lbs.
1	5/8"	5/8"	.320	1.09	.10	1.79	14"	16	18.7	33.4
2	5/8"	11/16"	.343	1.17	.10	1.91	14"	8	9.4	18.0
3	11/16"	11/16"	.367	1.25	.10	2.03	14"	32	37.3	75.7
4	15/16"	15/16"	.570	1.94	.15	3.14	16"	16	21.3	67.0
5	15/16	1"	.598	2.04	.15	3.28	16"	40	53.4	175.2
6	1 1/2"	1 1/2"	1.100	3.74	.20	5.90	16"	24	32.0	188.8
7	1 1/2"	1 3/8"	1.166	3.96	.20	6.23	16"	24	32.0	199.4
8	1 1/2"	1 3/4"	1.236	4.20	.20	6.60	16"	24	32.0	211.2
9	1 3/8"	1 3/4"	1.318	4.48	.20	7.02	16"	96	128.0	898.6
							TOTAL	280	364.1	1867.3
FIELD BUTT WELDS IN WEB PLATES 5/16" Pl. to 3/8" Pl.									607.6	292
3/16" FILLETS ON TEE-BUTT WELDS AT FIELD SPLICES									560	56
TOTAL WEIGHT OF FIELD WELD METAL (IN LBS)										2,215

SHOP WELDING

SIZE AND TYPE OF WELD	TOTAL LENGTH IN LIN. FT.	TOTAL WEIGHT IN LBS.
Tee-Butt welds with 3/16" Fillets. Flg Pl. to Web Pl.	25,800	2,580
Tee-Butt welds, 1/4" Fillets. Flg. Pls. to 7/16 Web Pls.	5,600	1,120
3/16 Intermittent fillets on bearing stiffeners	963.5	121
3/16 Intermittent fillets on 4 x 5/16 Stiffeners	12,322	1,555
Butt welds 5/16 to 5/16 web plates	57.6	16
Butt welds 3/8 to 7/16 web plates	72.1	448
3/8 Fillets welding stiff. bars to top flange	1,542	555
TOTAL WEIGHT OF SHOP WELD METAL IN LBS		6,395

ESTIMATED STRUCTURAL STEEL

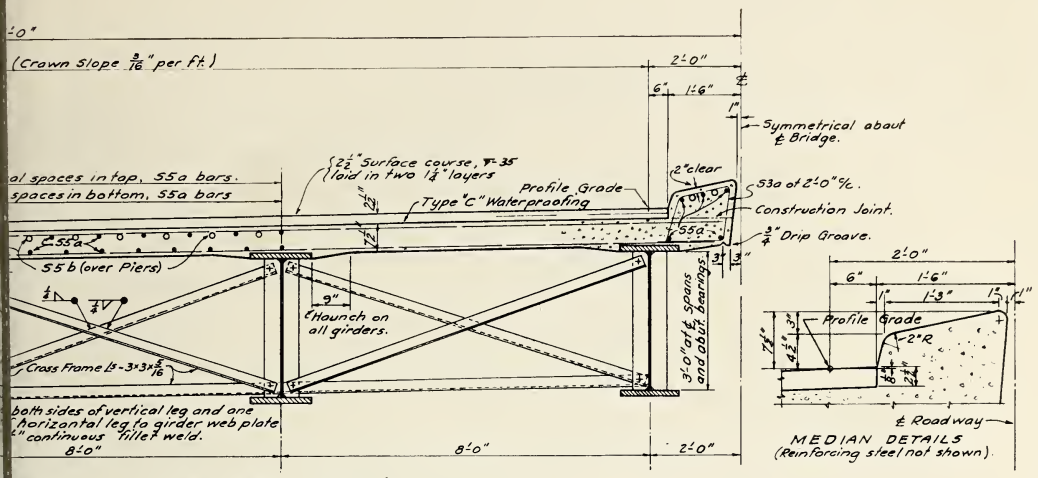
PLATE SIZE	TOTAL No. Pcs.	TOTAL WEIGHT
5/16" Web Plates	100	237,170
3/8" Web Plates	140	88,680
7/16" Web Plates	70	143,950
SUB-TOTAL	310	469,800
14 x 3/8 Flg. Plates	24	39,750
14 x 11/16 Flg. Pls.	36	60,880
16 x 5/16 Flg. Pls.	40	125,260
16 x 1 Flg. Pls.	20	40,260
16 x 1 3/8 Flg. Pls.	72	424,320
16 x 1 1/2 Flg Pls.	48	210,830
16 x 1 3/4 Flg. Pls	60	211,340
SUB-TOTAL	300	1,112,640
4 x 5/16 Stiff. Bars	2,644	44,840
5 x 5/16 Stiff. Bars	8	128
6 x 3/8 Stiff. Bars	32	732
7 x 3/8 Stiff. Pls.	28	1,500
8 x 1/2 Stiff. Pls.	28	2,290
8 x 3/8 Stiff. Pls.	84	8,570
SUB-TOTAL	2,824	58,060
50% (2.5%) over-run		20,500
GRAND TOTAL	3,434	1,661,000

EST...

ITEM
A2 AND B2
5-1
5-3
5-4
5-14
5-29
7-35
SUB
A2
5-7
5-8
SUB
GRA...
B2
Iron... Ther...
5-7
5-8
SUB
GR...

COMPARISON OF STRUCTUR...

ITEM
Total deck weight in kips p.l.f.
TOTAL Structural steel, Item 5-7.
Structural steel in the girders only
Rivet heads for Exhibit A2; Weld met...
Rivet heads or weld metal per ton of g...
Estimated cost of girders (life...
Estimated total superstructure
Total gallons of field paint on
Average weight of girders p.l.f.



ECTION (AT CENTER OF SPAN)

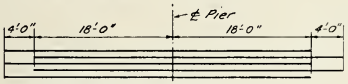


DIAGRAM SHOWING STAGGER OF 55b BARS OVER PIERS ~

STIMATED SUPERSTRUCTURE COSTS - EXHIBITS A2 AND B2

M	DESCRIPTION	ESTIMATED QUANTITY	UNIT	EST. UNIT COST $	ESTIMATED ITEM COST
	Class "C" Concrete, superstructure	1,560	cu yd	57.50	89,700
	Type "C" waterproofing	4,700	sq yd	1.00	4,700
	Reinforcing steel	403,660	lb	.11	44,403
	Railing (steel) (40 lb p.l.f.)	1,598	lin. ft.	15.00	23,970
	Subdrainage for wearing surface	1,594	lin. ft.	1.25	1,993
	Asphaltic concrete surface course	321	cu. yd	26.00	8,346
	B-TOTAL FOR ABOVE ITEMS (SAME FOR EXHIBITS A2 AND B2) =				$ 173,112
	Structural Steel	2,528,000	lb.	.1825	461,360
	Field Painting struct. steel (2 coats)	2,528,000	lb.	.01	25,280
	B-TOTAL FOR ITEMS S-7 AND S-8 FOR EXHIBIT A2 =				$ 486,640
	AND TOTAL SUPERSTRUCTURE COST FOR EXHIBIT A2 =				$ 659,752
	TEMS S-1, S-3, S-4, S-19, S-29 and T-35 are the same for Exhibits A2 & B2 refore sub-total cost for above items are estimated as				$ 173,112
	Structural Steel	1,863,600	lb.	.1350	251,586
	Field Painting struct. steel (2 coats)	1,863,600	lb.	.01	18,636
	B-TOTAL FOR ITEMS S-7 AND S-8 FOR EXHIBIT B2 =				$ 270,222
	AND TOTAL SUPERSTRUCTURE COST FOR EXHIBIT B2 =				$ 443,334

RAL AND COST DATA - EXHIBITS A2 & B2

	EXHIBIT A2	EXHIBIT B2	REDUCTION AMOUNT	REDUCTION %
deck	12.68	11.84	.84	6.65
(in tons)	1,264.0	931.8	332.2	26.3
(in tons)	1,167.0	834.8	332.2	28.5
tal B2 (lbs)	49,560	8,610	40,950	82.5
girder (lbs)	42.5	10.3	32.2	75.8
m S-7)	$461,360	251,586	209,774	45.5
cost	$659,752	443,334	216,418	32.9
girders	407	277	130	32.9
bridge (lb.)	2,920	2,090	830	28.5

NOTES.

The contractor or fabricator may make, at his own expense, any of the following three alterations to the girder construction.

1. Where the flange plates on the 5/16" web plates are of less thickness than the flange plates on the 3/8" and 7/16" web plates, the web depth of the 5/16" plate may be increased by an amount equal to the difference in thickness of the two flange plates in order that the field butt welds in the lower flange plates may be executed in the same flat position as the upper flange field butt welds.

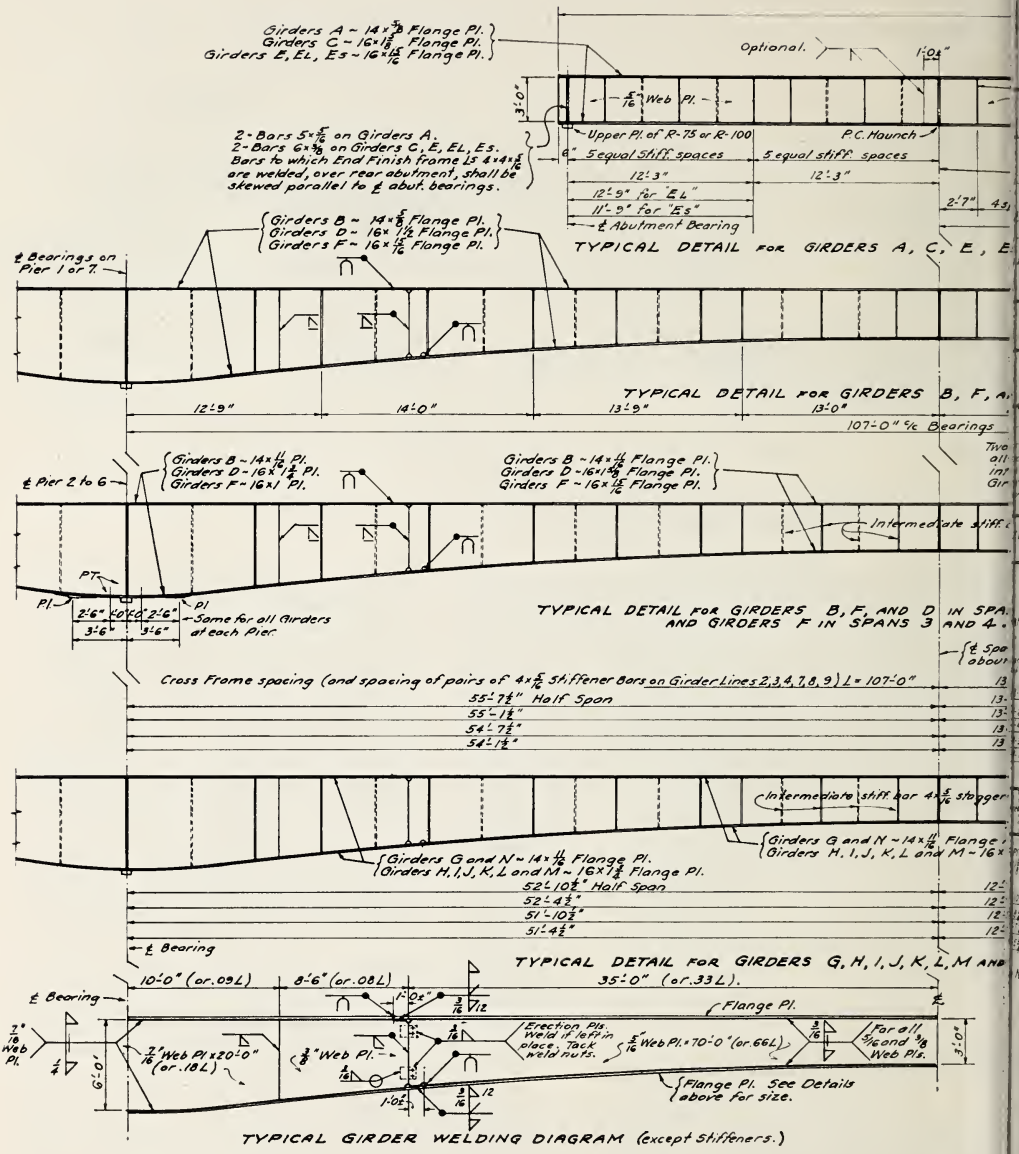
2. The field splices shown on the girder details may be made in the shop and a new field splice made in each girder over each pier without staggering flange and web splices. This alteration will require 2275 lbs. more shop weld metal, but field weld metal and number of splices will be reduced 50% (i.e. from 2215 lbs. to approx 1100 lbs and from 140 splices to 70 splices.

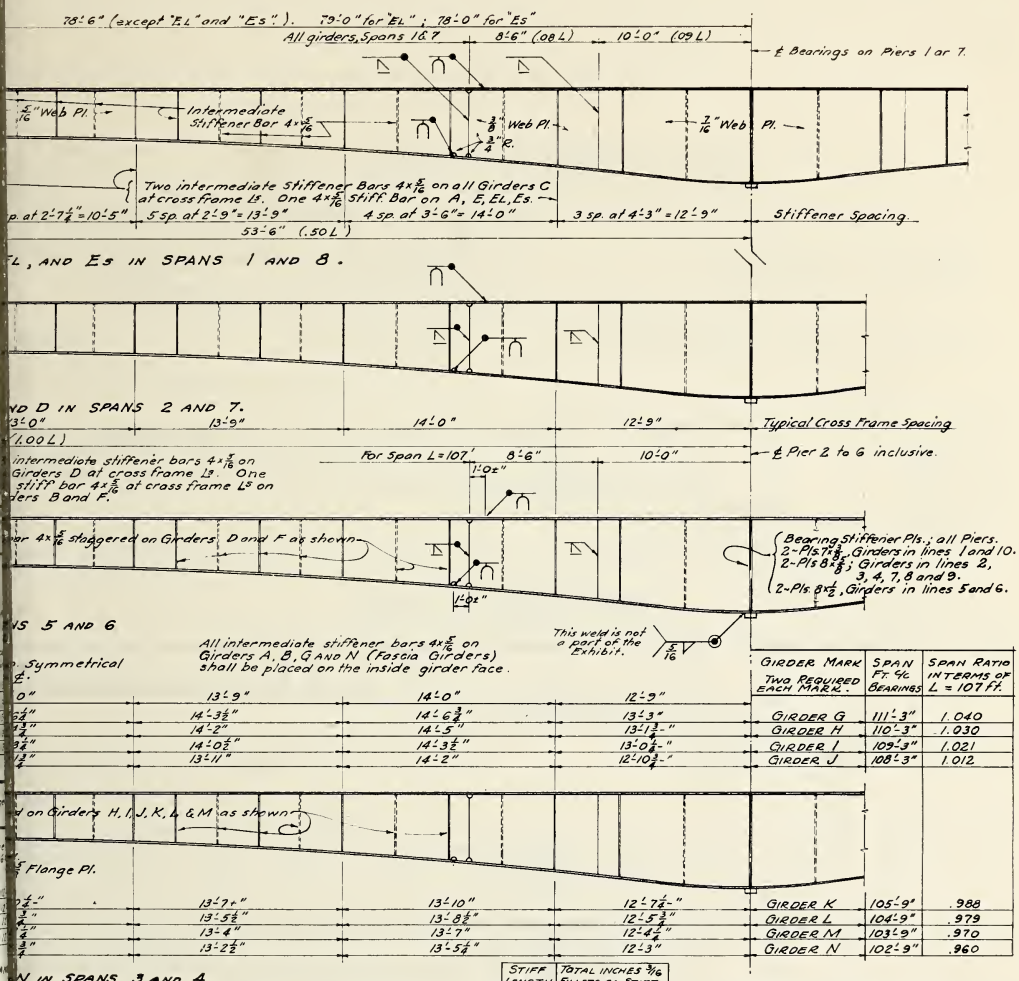
3. A 7/16" web plate may be used instead of the 3/8" web plate shown on the details. This will require 14,780 lbs. more structural steel but save 721 lin ft. of shop butt welding requiring 448 lbs. of weld metal where 3/8" and 7/16" webs were joined. However, welding the 7/16" web to the 5/16" web will and 85 lbs. more weld metal.

SHOP WELD METAL for the structural steel in the girders is estimated to be 7.65 lbs. per ton (finished weight).

FIELD WELD METAL for the structural steel in the girders is estimated to be 2.65 lbs. per ton (finished weight = 834.8 tons).

Figure 9


Girders A ~ 14 x 5/8 Flange Pl.
Girders C ~ 16 x 1 1/2 Flange Pl.
Girders E, EL, Es ~ 16 x 15/16 Flange Pl.
Optional.
1'-0"
3'-0"
5/16 Web Pl.
2 - Bars 5 x 5/16 on Girders A.
2 - Bars 6 x 3/8 on Girders C, E, EL, Es.
Bars to which End Finish frame Ls 4 x 4 x 5/16 are welded, over rear abutment, shall be skewed parallel to ℄ abut. bearings.
Upper Pl of R-75 or R-100
P.C. Haunch
6" 5 equal Stiff. spaces
5 equal stiff. spaces
12'-3"
12'-3"
12'-9" for "EL"
11'-9" for "Es"
2'-7"
℄ Abutment Bearing
TYPICAL DETAIL FOR GIRDERS A, C, E, E
Girders B ~ 14 x 5/8 Pl.
Girders D ~ 16 x 1/2 Flange Pl.
Girders F ~ 16 x 15/16 Flange Pl.
℄ Bearings on Pier 1 or 7
TYPICAL DETAIL FOR GIRDERS B, F, A
12'-9"
14'-0"
13'-9"
13'-0"
107'-0" c/c Bearings
℄ Pier 2 to 6
Girders B ~ 14 x 11/16 Pl.
Girders D ~ 16 x 1 1/2 Pl.
Girders F ~ 16 x 1 Pl.
Girders B ~ 14 x 11/16 Flange Pl.
Girders D ~ 16 x 15/16 Flange Pl.
Girders F ~ 16 x 15/16 Flange Pl.
Intermediate Stiff.
PT
Pl
2'-6"
3'-6"
40'-0" c/c
2'-6"
3'-6"
Pl
Same for all Girders at each Pier.
TYPICAL DETAIL FOR GIRDERS B, F, AND D IN SPA
AND GIRDERS F IN SPANS 3 AND 4.
Cross Frame spacing (and spacing of pairs of 4 x 5/16 Stiffener Bars on Girder Lines 2, 3, 4, 7, 8, 9) L = 107'-0"
55'-7 1/2" Half Span
55'-1 1/2"
54'-7 1/2"
54'-1 1/2"
Intermediate stiff. bar 4 x 5/16 stagger
Girders G and N ~ 14 x 11/16 Flange Pl.
Girders H, I, J, K, L and M ~ 16 x 1 1/2 Flange Pl.
52'-10 1/2" Half Span
52'-4 1/2"
51'-10 1/2"
51'-4 1/2"
℄ Bearing
TYPICAL DETAIL FOR GIRDERS G, H, I, J, K, L, M AND
℄ Bearing
10'-0" (or .09L)
8'-6" (or .08L)
35'-0" (or .33L).
1'-0"
3/16
7/12
Flange Pl.
7/16 Web Pl.
6'-0"
2 7/16 Web Pl. x 20'-0" (or .18L)
3/8 Web Pl.
Erection Pls. Weld if left in place. Tack weld nuts.
5/16 Web Pl. x 70'-0" (or .66L)
3/16
For all 5/16 and 3/8 Web Pls.
3'-0"
1'-0 1/2"
3/16
7/12
Flange Pl. See Details above for size.
TYPICAL GIRDER WELDING DIAGRAM (except Stiffeners.)


78'-6" (except "EL" and "Es"). 79'-0" for "EL"; 78'-0" for "Es"
All girders, Spans 1 & 7
8'-6" (.08 L)
10'-0" (.09 L)
℄ Bearings on Piers 1 or 7.
Intermediate Stiffener Bar 4x 5/16
3/8 Web Pl.
7/16 Web Pl.
Two intermediate Stiffener Bars 4x 5/16 on all Girders C at cross frame Ls. One 4x 5/16 Stiff. Bar on A, E, EL, Es.
3 sp. at 4'-3" = 12'-9"
Stiffener Spacing
53'-6" (.50 L)
L, AND Es IN SPANS 1 AND 8.
ND D IN SPANS 2 AND 7.
13'-0"
13'-9"
14'-0"
12'-9"
Typical Cross Frame Spacing
(1.00 L)
intermediate stiffener bars 4x 5/16 on Girders D at cross frame Ls. One stiff. bar 4x 5/16 at cross frame Ls on ers B and F.
for Span L = 107'
8'-6"
10'-0"
℄ Pier 2 to 6 inclusive.
1'-0±"
ar 4x 5/16 staggered on Girders D and F as shown
Bearing Stiffener Pls.; all Piers.
2-Pls 7 3/8; Girders in lines 1 and 10.
2-Pls 8 5/8; Girders in lines 2, 3, 4, 7, 8 and 9.
2-Pls. 8x 1/2, Girders in lines 5 and 6.
1'-0±"
VS 5 AND 6
All intermediate stiffener bars 4x 5/16 on Girders A, B, C AND N (Fascia Girders) shall be placed on the inside girder face.
This weld is not a part of the Exhibit.
5/16
. Symmetrical
℄
0"
13'-9"
14'-0"
12'-9"
d on Girders H, I, J, K, L & M as shown
5/8 Flange Pl.
13'-7±"
13'-10"
12'-7±"
13'-5±"
13'-8±"
12'-5 3/4"
13'-4"
13'-7"
12'-4±"
13'-2±"
13'-5±"
12'-3"
N IN SPANS 3 AND 4.
7 spaces @ 4±" = 28±"
7 " " " = 29 3/4"
8 " " " = 35 1/8
11 " " " = 46±"
14 sp. @ 4±" = 61±"
Stiff. bar 4x 5/16, stagger as shown
Top Flange
INTERMEDIATE STIFFENERS
Clip 1/2"
(See Spacing above)
close fit.
BEARING STIFFENERS
STIFFENER WELDING

GIRDER MARK (Two Required Each Mark)	SPAN Ft. & In. BEARINGS	SPAN RATIO IN TERMS OF L = 107 Ft.
GIRDER G	111'-3"	1.040
GIRDER H	110'-3"	1.030
GIRDER I	109'-3"	1.021
GIRDER J	108'-3"	1.012
GIRDER K	105'-9"	.988
GIRDER L	104'-9"	.979
GIRDER M	103'-9"	.970
GIRDER N	102'-9"	.960

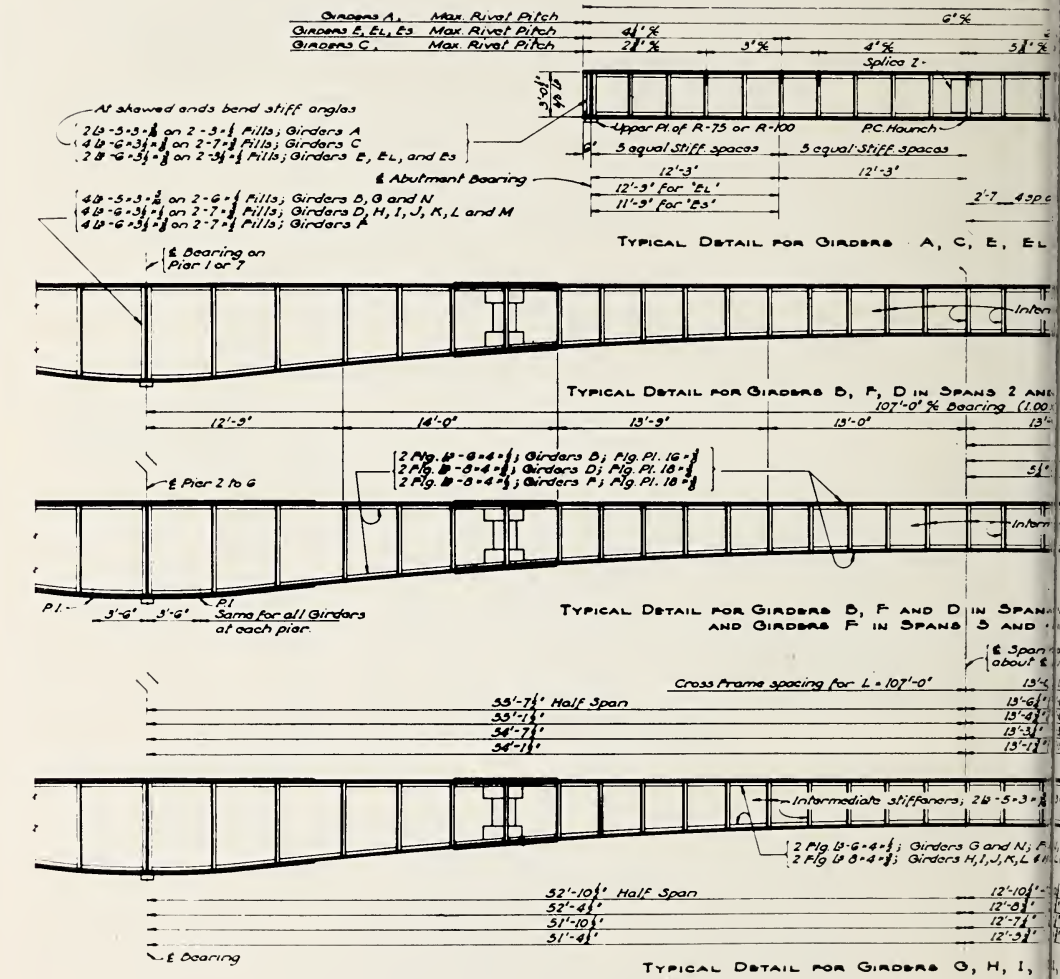
STIFF.	LENGTH	TOTAL INCHES 3/16 FILLETS EA. STIFF.
3 3/4"	36"	34"
3 3/4"	37 1/2"	34"
3 3/4"	42 5/8±"	37"
3 3/4"	54±"	46"
3 3/4"	69±"	55"

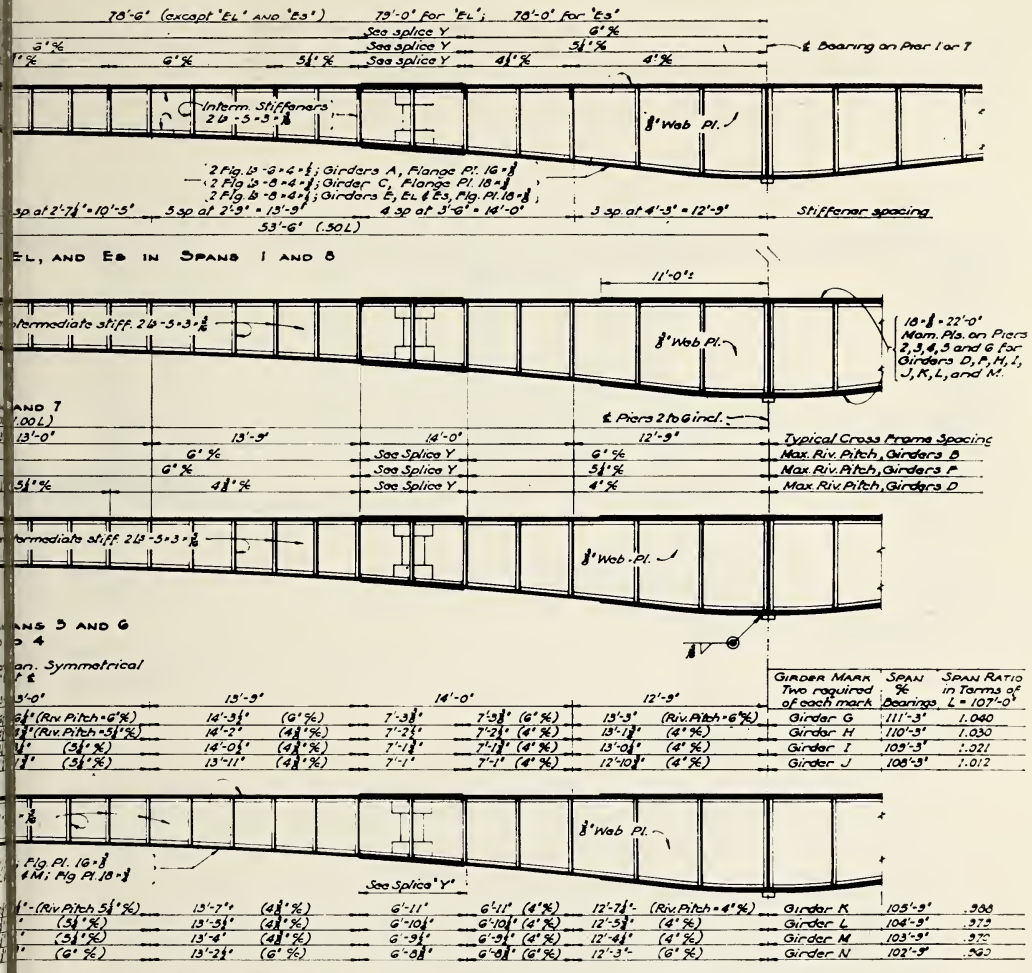
STIFF.	LENGTH	TOTAL INCHES EA. STIFF.	LOCATION
5x 5/16, 6x 7/16	4 1/2"	35"	At Abuts.
7x 3/8	8 1/2"	65"	lines 1 & 10
8x 1/2	9"	61"	lines 5 & 6
8x 5/8	9"	79"	2, 3, 4, 7, 8, 9

At Pier Brgs.

Diagonal cross frame angles in spans 3 and 4 are welded to stiffeners on Girders H, J, K and M.

Figure 10


Girders A, Max. Rivet Pitch
Girders E, EL, Es Max. Rivet Pitch
Girders C, Max. Rivet Pitch
6"
4½"
2½"
3"
4"
5½"
Splice L
At skewed ends bend stiff angles
2 Pl -5 x 3 - ⅜ on 2 - 3 x ½ Fills; Girders A
4 L -6 x 3½ x ⅜ on 2 - 7 x ⅜ Fills; Girders C
2 Pl -6 x 5½ x ⅜ on 2 - 3½ x ½ Fills; Girders E, EL, and Es
℄ Abutment Bearing
4 L -5 x 3 x 5/16 on 2 - 6 x ½ Fills; Girders B, G and N
4 L -6 x 3½ x ½ on 2 - 7 x ⅜ Fills; Girders D, H, I, J, K, L and M
4 L -6 x 5½ x ½ on 2 - 7 x ½ Fills; Girders F
Upper Pl. of R-75 or R-100
P.C. Haunch
5 equal Stiff. spaces
5 equal Stiff. spaces
12'-3"
12'-3"
12'-9" for 'EL'
11'-9" for 'Es'
2'-7
4 sp. @
TYPICAL DETAIL FOR GIRDERS A, C, E, EL
℄ Bearing on Pier 1 or 7
Intern
TYPICAL DETAIL FOR GIRDERS B, F, D IN SPANS 2 AND
107'-0" ℄ Bearing (1.00
12'-9"
14'-0"
13'-9"
13'-0"
2 Plg. L -6 x 4 x ½; Girders B; Plg. Pl. 16 x ⅜
2 Plg. L -6 x 4 x ½; Girders D; Plg. Pl. 18 x ⅜
2 Plg. L -6 x 4 x ½; Girders F; Plg. Pl. 18 x ⅜
℄ Pier 2 to 6
5½"
Intern
P1
3'-6"
3'-6"
P1
Same for all Girders at each pier.
TYPICAL DETAIL FOR GIRDERS B, F AND D IN SPAN
AND GIRDERS F IN SPANS 5 AND
℄ Span about ℄
Cross Frame spacing for L = 107'-0"
13'-
55'-7½" Half Span
55'-1½"
54'-7½"
54'-1½"
13'-6½"
13'-4½"
13'-3½"
13'-1½"
Intermediate stiffeners; 2 L -5 x 3 x
2 Plg. L -6 x 4 x ½; Girders G and N; F
2 Plg. L 6 x 4 x ⅜; Girders H, I, J, K, L &
52'-10½" Half Span
52'-4½"
51'-10½"
51'-4½"
12'-10½"
12'-8½"
12'-7½"
12'-5½"
℄ Bearing
TYPICAL DETAIL FOR GIRDERS G, H, I,


78'-6" (except 'E1' and 'E3')
79'-0" for 'E1'; 78'-0" for 'E3'
See splice Y
℄ Bearing on Pier 1 or 7
Interm. Stiffeners 2 L - 5 - 5/8
5/8" Web Pl.
2 Flg. Pls. -6 x 4 x 1/2; Girders A, Flange Pl. 16 x 3/4
2 Flg. Pls. -8 x 4 x 1/2; Girder C, Flange Pl. 18 x 3/4
2 Flg. Pls. -8 x 4 x 1/2; Girders B, E1 & E3, Flg. Pl. 18 x 3/4
3 sp. at 2'-7 1/2" = 10'-5"
3 sp. at 3'-3" = 13'-9"
4 sp. at 3'-6" = 14'-0"
3 sp. at 4'-3" = 12'-9"
Stiffener spacing
53'-6" (50 L)
E1, and E3 in Spans 1 and 8
11'-0"±
Intermediate stiff. 2 L - 5 x 3 x 5/16
5/8" Web Pl.
18 x 3/4 - 22'-0" Nom. Pls. on Piers 2, 3, 4, 5 and 6 for Girders D, F, H, I, J, K, L, and M.
℄ Piers 2 to 6 incl.
13'-0"
13'-9"
14'-0"
12'-9"
Typical Cross Frame Spacing
Max. Riv. Pitch, Girders B
Max. Riv. Pitch, Girders F
Max. Riv. Pitch, Girders D
Intermediate stiff. 2 L - 5 x 3 x 5/16
5/8" Web Pl.
Spans 3 and G
Sym. Symmetrical
See Splice 'Y'
Flg. Pl. 16 x 3/4
Web Pl. 5/8
Girder K
Girder L
Girder M
Girder N
J, K, L, M and N in Spans 3 and 4

Girder Mark Two required of each mark	Span % Bearings	Span Ratio in Terms of L = 107'-0"
Girder G	111'-3"	1.040
Girder H	110'-3"	1.030
Girder I	109'-3"	1.021
Girder J	108'-3"	1.012
Girder K	105'-9"	.988
Girder L	104'-9"	.979
Girder M	103'-9"	.970
Girder N	102'-9"	.960

Rivets shall be 7/8"ϕ

See Sheets No 154, 156, 157, 159, 160, 161 for further details.

Figure 11

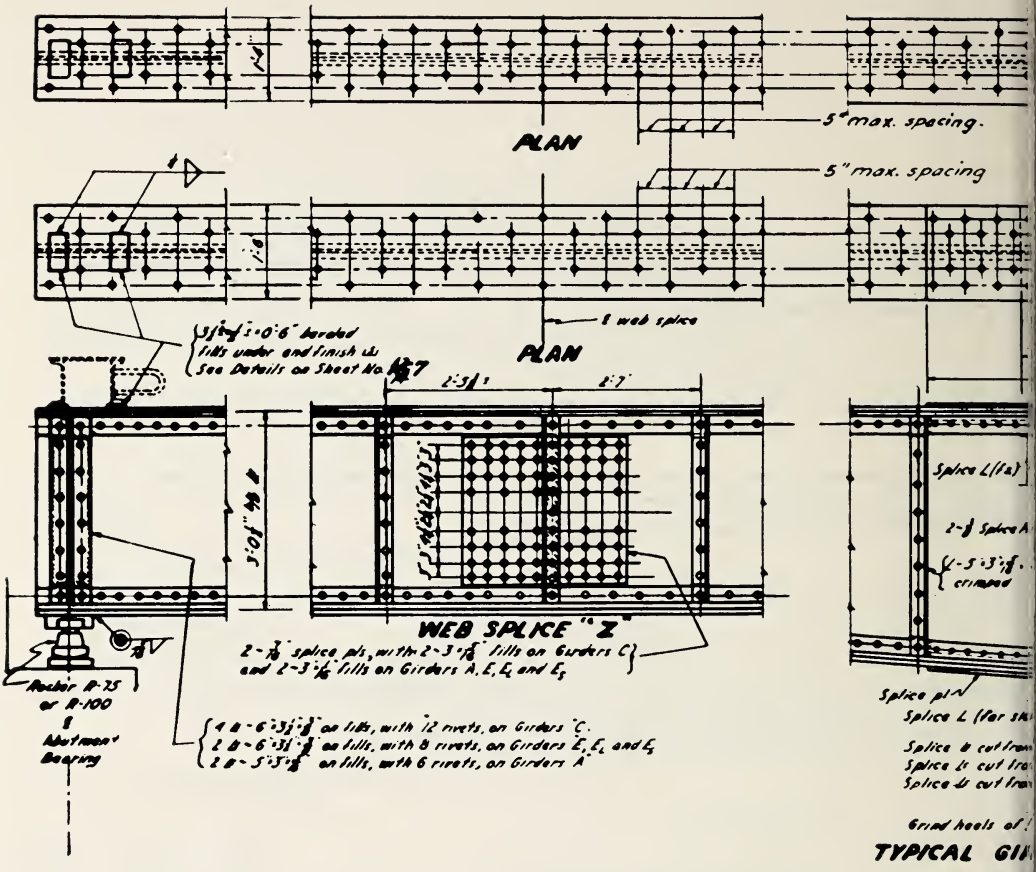


Figure 12

"All of the design calculations for Exhibit A2 were used for Exhibit B2, except the computations relating to riveting, stiffeners and splicing. Exhibit B2 required new designs for girder sections and stiffener sections. The spacing of intermediate stiffeners was made the same for both exhibits, but were used in pairs on Exhibit A2 and singly on B2, except where required in pairs for cross frame connections. [Elsewhere in his text, the author said: 'It requires no more holes in the web plate, or rivets, for a pair of stiffener angles than for a single one. However, the specifications permit single stiffeners, and a pair of welded stiffener plates require twice as much material and labor as a single one. Therefore there is more of a saving when welded stiffener plates are used singly. Welded stiffeners will be used in pairs only where required over bearings and for connections for cross frame angles'.] The single inter-

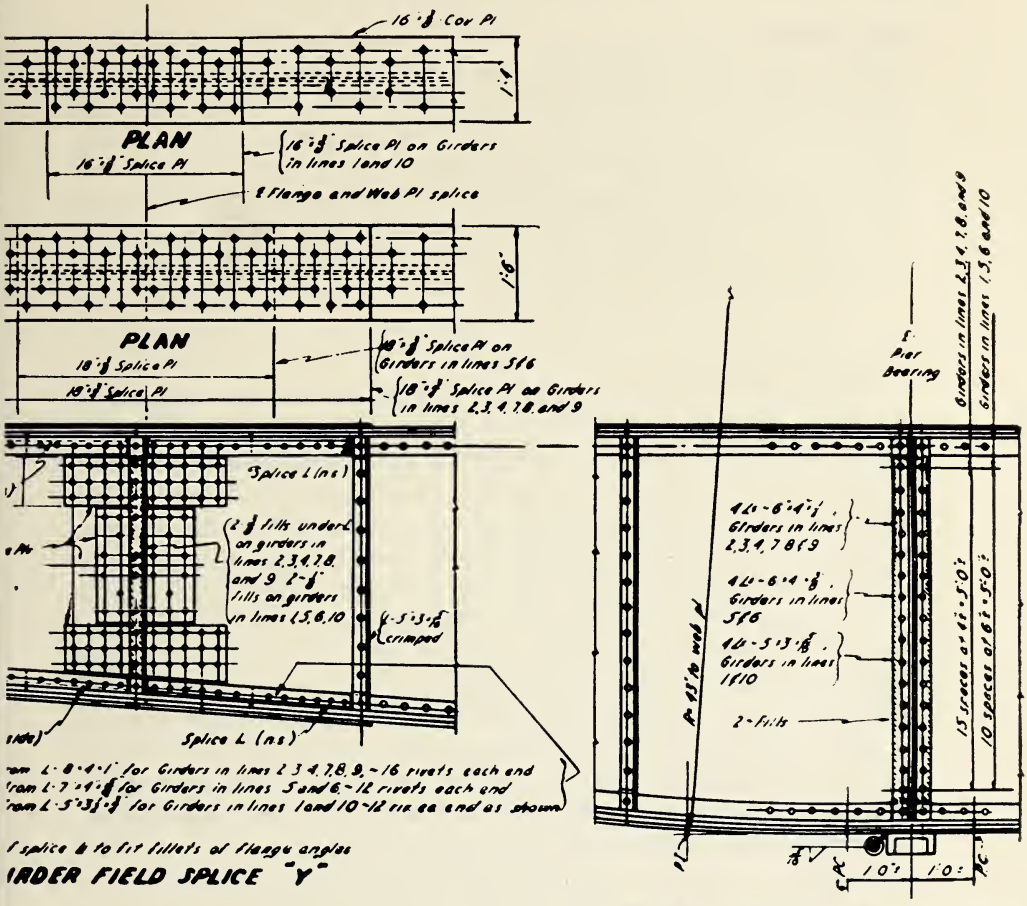


Figure 12

mediate stiffeners were staggered except on the fascia girders. . . .

"There was a change in the web design for Exhibit B2. The web plate thickness was made ⅜ in. throughout on the riveted girders (Exhibit A2) even where clear, unsupported web depths of less than 53 in. permitted use of 5/16 in. thickness. There were two advantages in maintaining a ⅜ in. web plate throughout on the riveted girders. The main advantage was the rivet value. The bearing value of a ⅞ in. rivet is 8,860 lb. on a ⅜ in. web and only 7,383 lb. on a 5/16 in. web, which would mean a reduction of 17 per cent in value that could only be regained by using more rivets. The cost and weight of the increased amount of rivets would offset the economy of a thinner web, or else the section could remain as one 17 per cent weaker by reason of a 5/16 in. web. Where the web depth required a ⅜ in. thickness, the girder splice would need

fills, and this is not desirable. With a welded girder (Exhibit B2) the use of 5/16 in. web plates saved steel and reduced the cost of the Tee butt weld. The continuous butt weld develops the entire web section and seals the joint. The butt welding of 5/16 in. web plates to ⅜ in. web plates, and ⅜ in. to 7/16 in. webs presents no special detailing or fitting problems. 26,560 lbs. of structural steel were saved in Exhibit B2 by varying the web thickness, which represents a 5.35 per cent reduction in the weight of the web plates.

* * *

"The haunched riveted girders in Exhibit A2 required 1,167 tons of steel (including rivet heads), and Exhibit B2 834.8 tons (including weld metal). This is a savings of 332.2 tons, or 28.5 per cent in the structural steel in the girder. Breaking this savings down into the main members; 20.50 per cent was saved in the flanges, 5.35 per cent in the web plates, 63 per cent in the stiffeners, 100 per cent in the girder splices, and 90 per cent of rivet head weight versus gross fillet weld weights. Estimating ⅞ in. rivets at one pound each, the riveted girders required 159,630 lbs. of rivets versus 8,610 lbs. of weld metal; a reduction of 94.5 per cent. 107 lbs. of shop rivets were used per ton of steel for Exhibit A2 and 7.65 lbs. of field weld metal per ton for Exhibit B2. The weight of the rivet heads represented 2.125 per cent of the structural steel weight of the haunched girders of Exhibit A2, and the fillet welds on Exhibit B2 girders were .23 per cent of the girder weight. . . ."

See Figures 11 and 12 for details of the riveted girders.

Mr. Dumbauld included elaborate analyses of costs from which the following are taken:

The percentage distribution of structural steel cost analyzed as follows:

	Exhibit A2 Riveted	Exhibit B2 Welded Girders
1. Materials	22%	29%
2. Shop Fabrication	32%	12%
3. Freight	7%	9%
4. Social Security & Tax	4%	5%
5. Erection	26%	36%
6. Profit	9%	9%
Total	100%	100%

* * *

The preceding calculations and investigations have demonstrated the following advantages for Exhibit B2 over A2:

1. A saving of 332.2 tons of structural steel (28.5%)
2. A saving of $209,774 in the girder cost (45.5%)
3. A saving of $6,644 in paint cost (26.3%)
4. Saved 32% of the painting materials, which indicates a 32% savings in future maintenance paint costs
5. About 20% savings in time of designing and detailing
6. Estimate same savings in time (20%) for shop detailing and pattern making
7. Reduced tonnage for transportation and erection reduces wear and depreciation on transportation and equipment
8. Less noise in erecting
9. Greater flexibility in the manner of making shop and field splices permits greater variety in methods of shipping and erecting. This should create more competitive bidding and lower costs to the taxpayer.

"The preceding analysis of two modern deck girder highway bridges has demonstrated that there is substantial conservation of resources and wealth available by using welded construction in place of riveted construction. This is important as related in the two bridges designed and analyzed in this entry, but the entrant believes that an entry 'Dedicated to the National Interest' should contain a conscientious and proven statement as to the possible gross savings to a national program of highway construction and highway structures possible by use of welded plate girders when such a type of structure is deemed practical for a specified site.

"An attempt to make a correct estimate on the gross saving in steel and wealth made possible by welded bridges, as designed by the entrant, requires much reading, investigating and estimating. However, the entrant will make a statement as to the gross savings possible by the general adoption of his designs on similar bridges in the U. S. A. The remainder of this paper will be substantiating facts, tables, data and records to confirm his opinions.

"The estimated annual savings on several different twenty-year bridge construction programs using welded plate girders, designed similar to Exhibits B1, and B2, are analyzed on the remaining pages of this paper. The following estimated possible annual savings were predicted by the entrant:

1. 2900 tons per year on the State of Ohio Highway System.
2. 23,000 tons per year on the Federal Aid Highway System of the United States.
3. 73,000 tons per year on all public roads, streets and highways in the United States.
4. 40,000 tons in 1952 of the N. P. A. allotment of structural steel to the Bureau of Roads.

Milton D. Randall and Farland Bundy, both of Austin, Texas, designed a three span continuous girder bridge. The spans are 100 ft., 130 ft., and 100 ft. The welded girders have expanded webs, whereas, the riveted girders have solid webs. This difference is shown in Figure 13. The welded girders have solid webs and are haunched in the vicinity of the two center piers (see Figure 14). The 6½ in. reinforced concrete slab of the 28 ft. roadway is supported directly on four welded girders spaced at 8 ft. centers as the transverse section of Figure 15 shows.

The expanded portion of the girder is 4 ft.-5⅞ in. deep made by expanding a 36 WF 150. At the piers, the web is ⅜ in. thick and the overall depth is 7 ft.-10⅞ in. for the interior girders and 7 ft.-10½ in. for the exterior girders—the cover plates vary in thickness. Cover plates are added to the expanded beams in the middle of the end spans. Within the haunched part of the girders, the flange plates are 12 in. wide and ¾ in. thick, and near the piers 11 in. wide cover plates are added. Where the web is solid, both a longitudinal stiffener (3 x 3 x 5/16 angle) and vertical stiffeners (5 x ⅜ plates) are used. At each pier, two - 5½ x ¾ plates serve as the bearing stiffener. The diaphragms between girders are 9 ft. apart adjacent to the piers, but this spacing increases between piers.

Messrs. Randall and Bundy gave these reasons for selecting their structure.

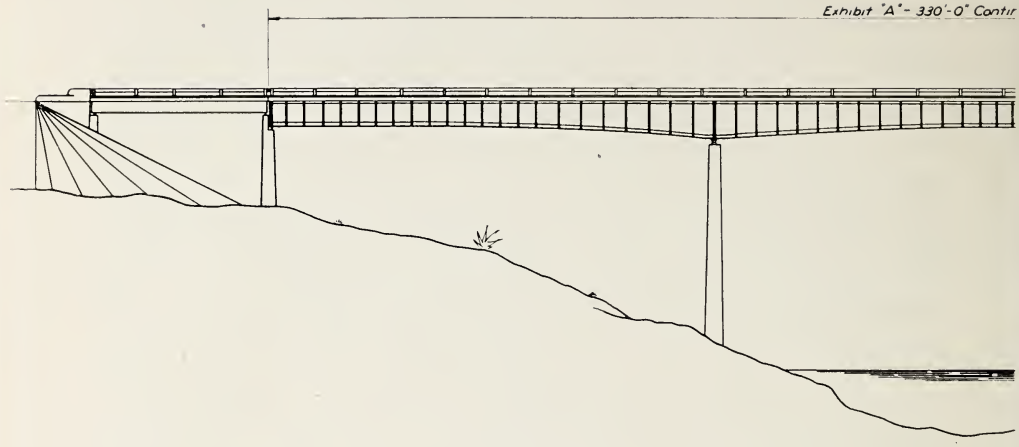
"The development of this comparison came about through the desire of the designers to make use of the expanded beam idea in a continuous girder unit and to compare it with a recently built riveted structure. The riveted structure selected was a deck plate girder unit with two curved soffit girders and a floorbeam and stringer system. An economic study indicated that the two girder scheme was preferred for riveted construction. This was not true, however, for welded construction. A four girder scheme was indicated here with a resulting saving in both structural steel weight and overall cost. For welded construction, the four girder scheme was preferred over the two-girder one whether or not the expanded

beam idea was employed. Therefore, the inability of the expanded beam to be used where a depth greater than about 60 inches is required was no limitation on its use in this welded structure. The designers feel that these two deck plate girder units are of 'similar structural types' as required by the rules of the contest and that a comparison between them is entirely justified. The increase in the number of girders to do away with the floor system of the riveted design is one of the important trends in welded bridge design today.

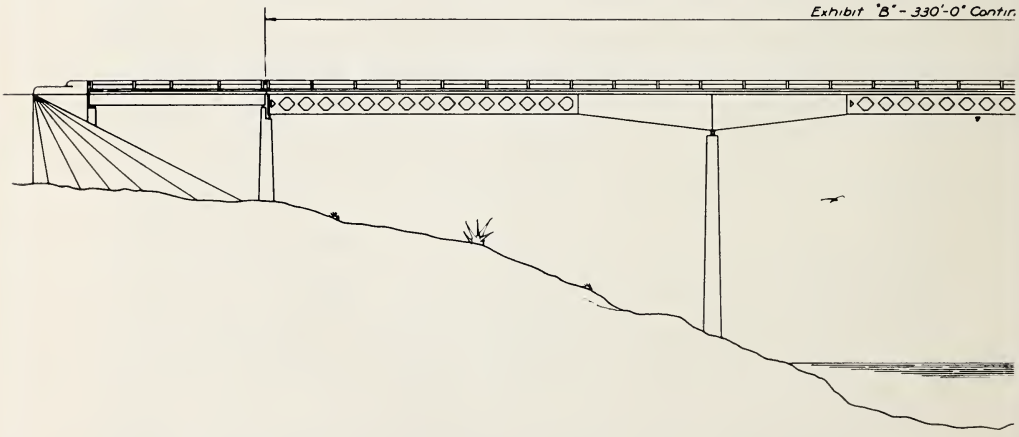
"The authors are familiar with the recent designs and fabrication of two bridges which employ the expanded beam idea. Both are simple spans of 100 feet and 65 feet, respectively. They were economical to fabricate and erect and present a very pleasing appearance. The expanded beam can be designed rather simply for both bending and shear resistence. There is a minimum of waste of material and a minimum of shop welding in the fabrication of an expanded beam section. The exceptionally small amount of shop welding more than offsets the cost of cutting. In this structure all 12 of the expanded beam sections are identical except for the bearing stiffeners required for sections in the end spans. The stress analyses for both the riveted and the welded structure were based on the use of influence lines which were arrived at by the application of the Müller-Breslau principle to the variable moment of inertia girders. Other features of the design follow the requirements of those specifications enumerated in the rules.

"The site of the bridge is over a small river with centerline profile and normal water elevation as indicated on the layout. Hydraulic studies indicate sufficient opening for passage of water and drift during flood stage. The low steel elevation is made several feet above the elevation of a seventy-five year frequency flood. The location of the substructure is determined by the character of the stream crossing, grade line and general cost studies. The short simple spans at each end of the girder units were present in the riveted structure as built and were thus indicated on the layout for both the riveted and the welded structure. They are not a part of this contest and are to be considered simply as approaches to the girder units. The site is a rural area where sidewalks are not required.

"The reinforcing steel and slab thickness are identical for both bridges. Essentially the same amount of forming would be required for placing of the concrete deck in both exhibits. The designers have some skepticism as to the desirability and economy of a participating floor in a unit where large areas of negative moment exist.



LAYOUT — EX



LAYOUT — EX

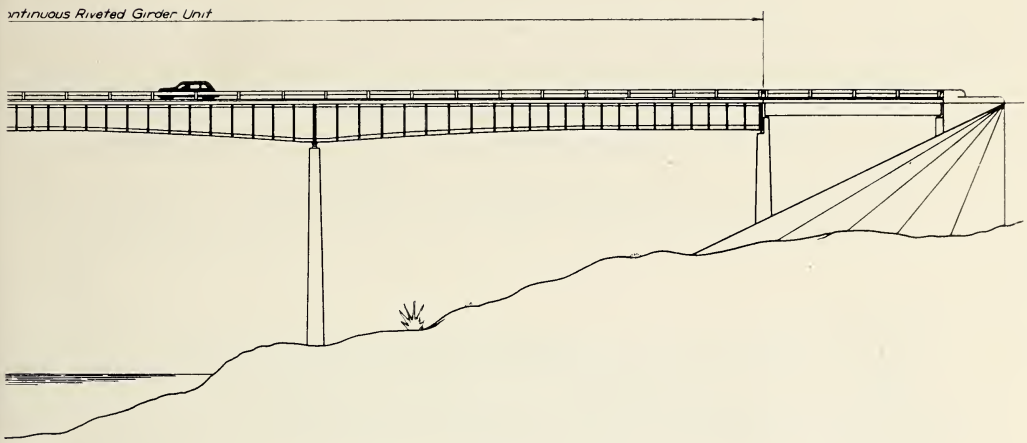


EXHIBIT "A"

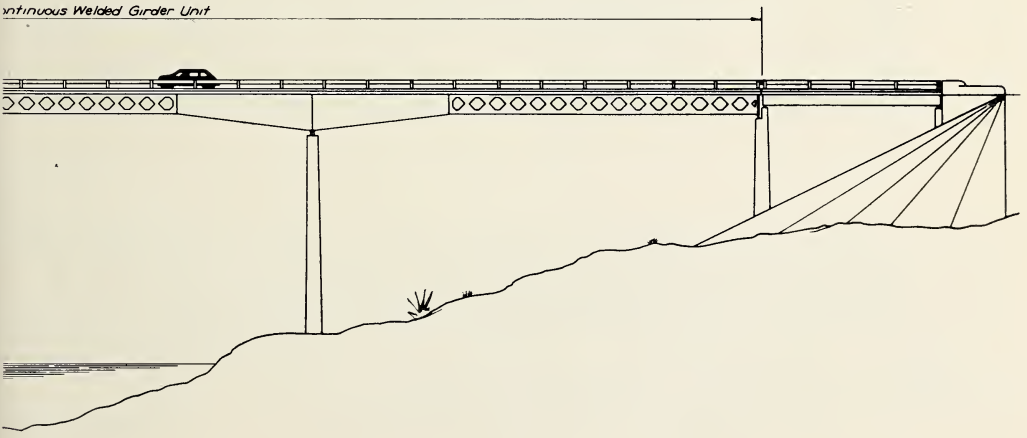
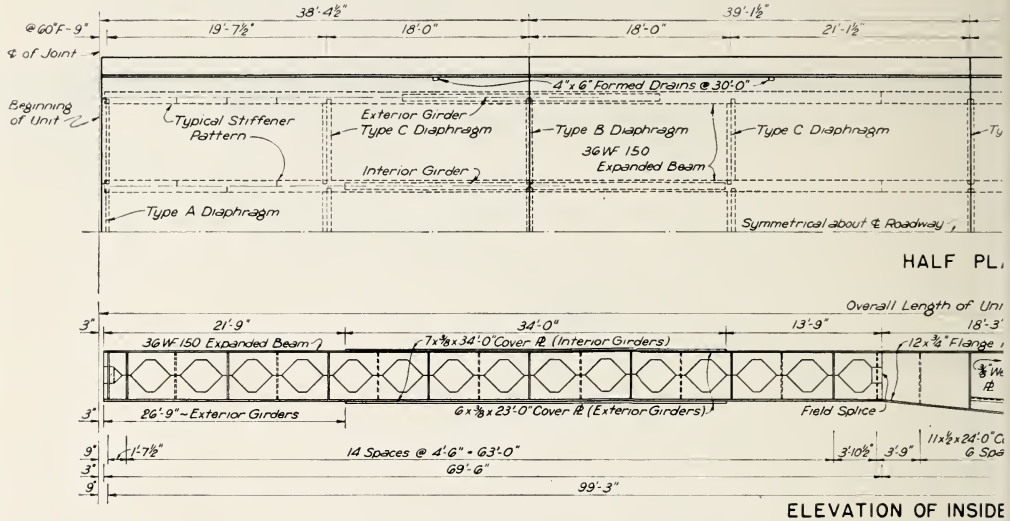


EXHIBIT "B"

Figure 13


38'-4½"
39'-1½"
@ 60°F - 9"
19'-7½"
18'-0"
18'-0"
21'-1½"
℄ of Joint
4"x6" Formed Drains @ 30'-0"
Beginning of Unit
Typical Stiffener Pattern
Exterior Girder
Type C Diaphragm
Type B Diaphragm
Type C Diaphragm
Interior Girder
36WF 150 Expanded Beam
Type A Diaphragm
Symmetrical about ℄ Roadway
HALF PL
Overall Length of Uni
3"
21'-9"
34'-0"
13'-9"
18'-3
36WF 150 Expanded Beam
7x⅜x34'-0" Cover Pl (Interior Girders)
12x⅞" Flange
⅜" We Pl
3"
26'-9" - Exterior Girders
6x⅜x23'-0" Cover Pl (Exterior Girders)
Field Splice
9"
1'-7½"
14 Spaces @ 4'-6" = 63'-0"
3'-10½"
3'-9"
11x⅛x24'-0" C
6 Spa
3"
69'-6"
9"
99'-3"
ELEVATION OF INSIDE
℄ Construction Jts
℄ Diaphragms
41'-4½"
22'-6"
22'-6"
18'-0"
23'-4½"
13'-6"
9'-0"
9'-0"
4"x6" Formed Drain
Exterior Girder
Type B Diaphragm
Type C Diaphragm
Type B Diaphragm
Type C Diaphragm
Type B Diaphragm
Type C Di
Match Line
Interior Girder
HALF PL
Overall Length of Unit = 330
20'-0"
24'-9"
18'-3"
24'-0"
L-3x3x⅝ Longitudinal Stiffener - One Side O
⅜" Web Pl
11½"
1'-6½"
Field Splice
℄ Bearing
5½x⅜ Pl Stif
℄ ℄ Stiffeners
14 Spaces @ 4'-6" = 63'-0"
5'-4½"
6 Spaces @ 4'-6" = 27'-0"
6 Spaces @ 4'-
℄ ℄ Field Splices
69'-6"
60'-6"
℄ ℄ Bearing
130'-0"
ELEVATION OF INSID

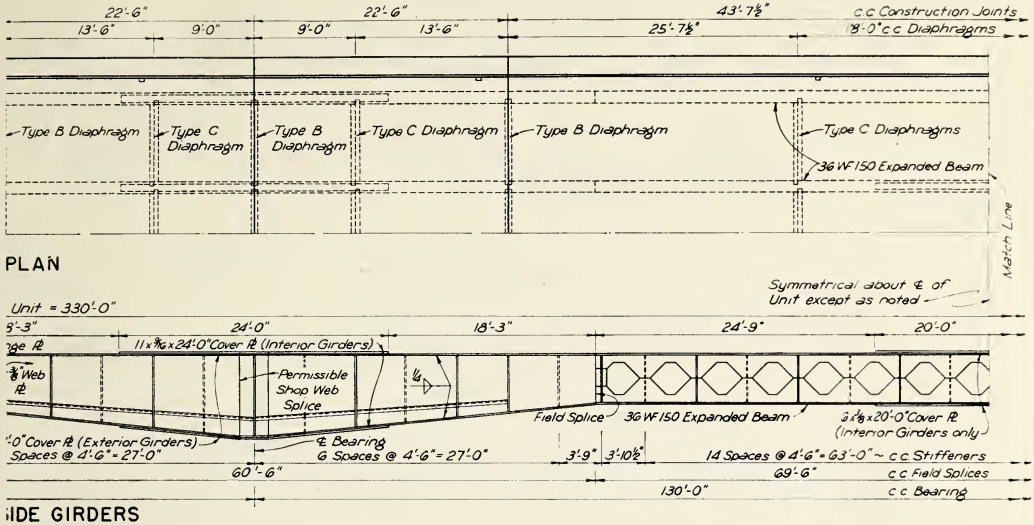
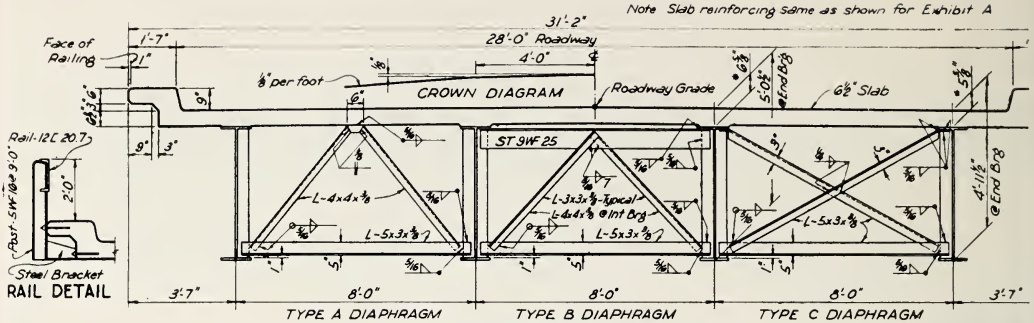
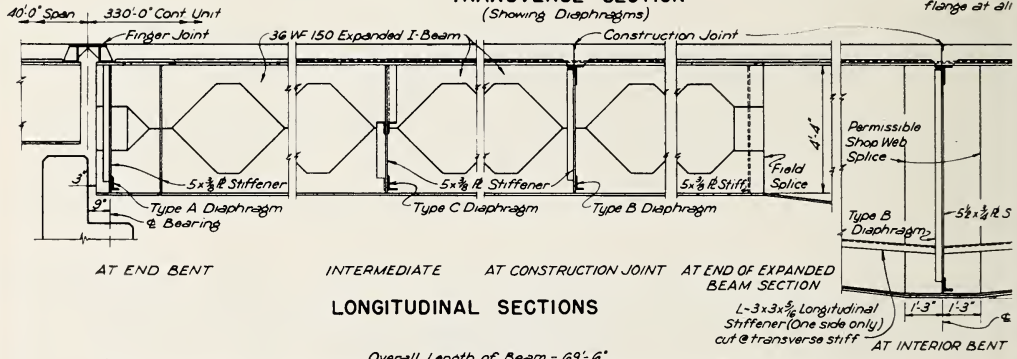
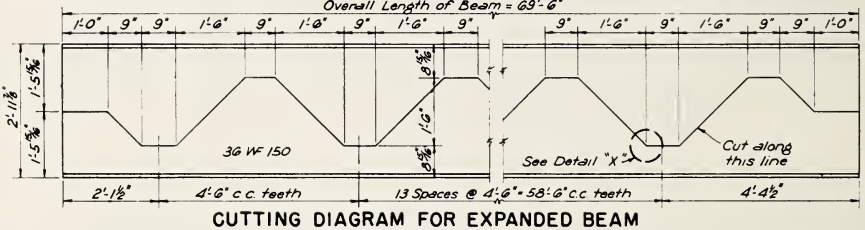
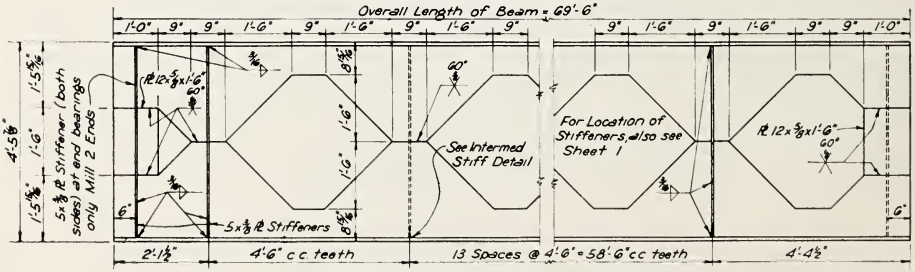

22'-6"
22'-6"
43'-7½"
c.c Construction Joints
13'-6"
9'-0"
9'-0"
13'-6"
25'-7½"
18'-0" c.c Diaphragms
Type B Diaphragm
Type C Diaphragm
Type B Diaphragm
Type C Diaphragm
Type B Diaphragm
Type C Diaphragms
36 WF 150 Expanded Beam
Match Line
PLAN
Symmetrical about ℄ of Unit except as noted
Unit = 330'-0"
24'-0"
18'-3"
24'-9"
20'-0"
11 x ⅞ x 24'-0" Cover Pl (Interior Girders)
Permissible Shop Web Splice
¼
Field Splice
36 WF 150 Expanded Beam
3 ⅛ x 20'-0" Cover Pl (Interior Girders only)
Spaces @ 4'-6" = 27'-0"
℄ Bearing
6 Spaces @ 4'-6" = 27'-0"
3'-9"
3'-10½"
14 Spaces @ 4'-6" = 63'-0" c.c Stiffeners
60'-6"
69'-6"
c.c Field Splices
130'-0"
c.c Bearing
SIDE GIRDERS
℄ of Joint
39'-1½"
38'-4½"
13'-6"
21'-1½"
18'-0"
18'-0"
19'-7½"
End of Unit
Type B Diaphragm
Type C Diaphragm
Type B Diaphragm
Type C Diaphragm
Type A Diaphragm
Symmetrical about ℄ Roadway
PLAN
330'-0"
18'-3"
13'-9"
34'-0"
21'-9"
3"
Field Splice
5 x ⅜ Pl Stiffeners (Alternate Sides (Typical)
5 x ⅜ Pl Stiffeners - Both Sides @ Diaphragms Only (Typical)
5 x ⅜ Pl Stiffener - Both Sides
3'-9"
3'-10½"
14 Spaces @ 4'-6" = 63'-0"
1'-7½"
9"
69'-6"
3"
99'-3"
9"
SIDE GIRDERS
Note On Exterior Girders, Intermediate Stiffeners and Longitudinal Stiffeners shall be only on inside.
Exterior Girder same as Interior Girder except as noted

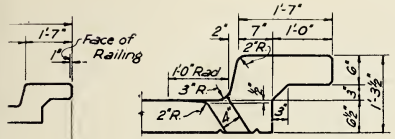
Figure 14


Note Slab reinforcing same as shown for Exhibit A
31'-2"
28'-0" Roadway
4'-0"
Face of Railing
1'-7"
21"
⅛" per foot
CROWN DIAGRAM
Roadway Grade
6½" Slab
5'-0" at End Brg
Rail-12# 20.7
Post-SW-Use 3'-0"
2'-0"
Steel Bracket
RAIL DETAIL
ST 9WF 25
L-4x4x⅜
L-5x3x⅜
L-3x3x⅜-Typical
L-4x4x⅜ @ Int Brg
4'-11½" @ End Brg
3'-7"
8'-0"
TYPE A DIAPHRAGM
TYPE B DIAPHRAGM
TYPE C DIAPHRAGM
TRANSVERSE SECTION
(Showing Diaphragms)
*Depth of slab flange at abutment


40'-0" Span
330'-0" Cont Unit
Finger Joint
36 WF 150 Expanded I-Beam
Construction Joint
5x⅜ PL Stiffener
Type A Diaphragm @ Bearing
Type C Diaphragm
Type B Diaphragm
Field Splice
Permissible Shop Web Splice
Type B Diaphragm
AT END BENT
INTERMEDIATE
AT CONSTRUCTION JOINT
AT END OF EXPANDED BEAM SECTION
L-3x3x5/16 Longitudinal Stiffener (One side only) cut @ transverse stiff
AT INTERIOR BENT
LONGITUDINAL SECTIONS

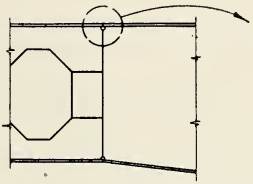

Overall Length of Beam = 69'-6"
36 WF 150
See Detail "X"
Cut along this line
2'-1½"
4'-6" c.c. teeth
13 Spaces @ 4'-6" = 58'-6" c.c. teeth
4'-4½"
5x⅜ PL Stiffener (Mill 2 Ends)
CUTTING DIAGRAM FOR EXPANDED BEAM


Overall Length of Beam = 69'-6"
PL 12x⅝x1'-6"
60°
5x⅜ PL Stiffeners
5x⅜ PL Stiffener (both sides) at end bearings only Mill 2 Ends
See Intermed Stiff Detail
For Location of Stiffeners, also see Sheet 1
2'-1½"
4'-6" c.c. teeth
13 Spaces @ 4'-6" = 58'-6" c.c. teeth
4'-4½"
ASSEMBLY DIAGRAM FOR EXPANDED BEAM

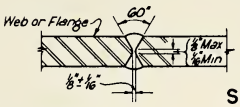


CURB AND DRAIN DETAIL

4"x 6" Formed Drain. Trowel entrance to give ½" entrance depression. Provide ½" drip bead around drain. Bend Reinforcing steel to clear drain 1"



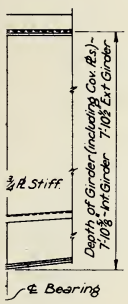
Note. Ends of girders to be welded shall be prepared in the shop and each splice checked by assembling the two beam sections to be spliced and the pieces matchmarked



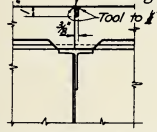
SPLICE DETAILS

Note. Sequence of welding field splices shall be approved by the Engineer

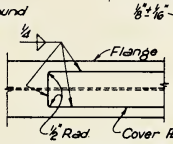
slab over top
it all bearings.



ENT



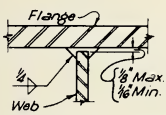
CONSTRUCTION JOINT DETAIL



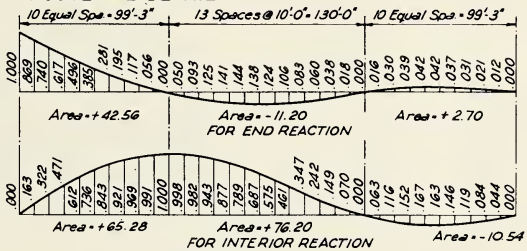
TYP. COVER PL. DETAIL

	End Span	Int Span
Calculated	0.96"	1.55"
Allowable	1.49"	1.95"

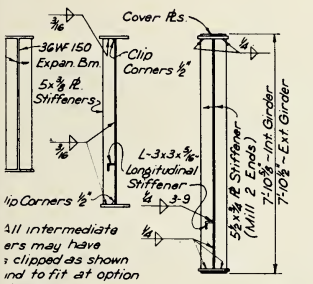
DEFLECTIONS FOR LIVE LOAD + IMPACT



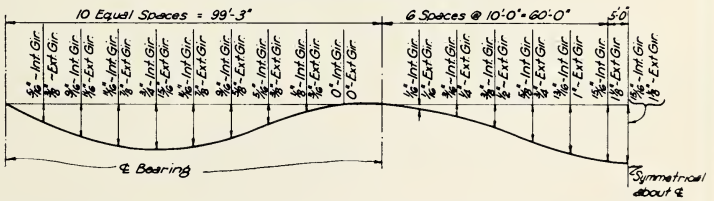
WEB TO FLANGE CONN. DETAIL



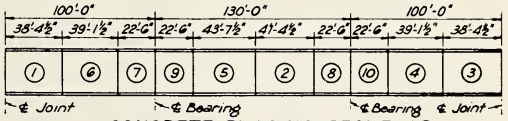
INFLUENCE LINES



STIFFENER DETAILS



DEAD LOAD DEFLECTION DIAGRAM FOR CONC. AND RAILING ONLY



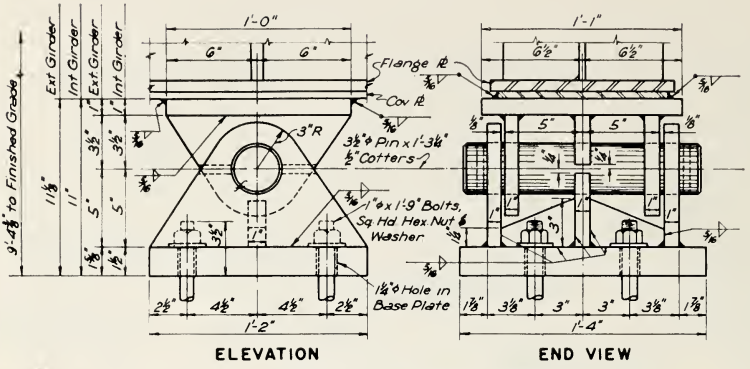
CONCRETE PLACING SEQUENCE

ETAIL "X"

eam shall be cut with
ine guided torch with
ous movement of torch
l corners of teeth.

Note: After all Structural Steel is erected, an accurate measurement shall be made of the elevations of the girder flanges at all control points. Subsequent setting of forms, placing and finishing of concrete shall be governed by these measurements only, taking into account the dead load deflections shown above.

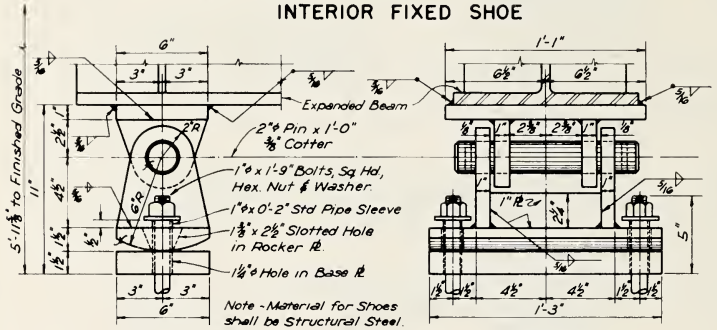
Figure 15



ELEVATION END VIEW

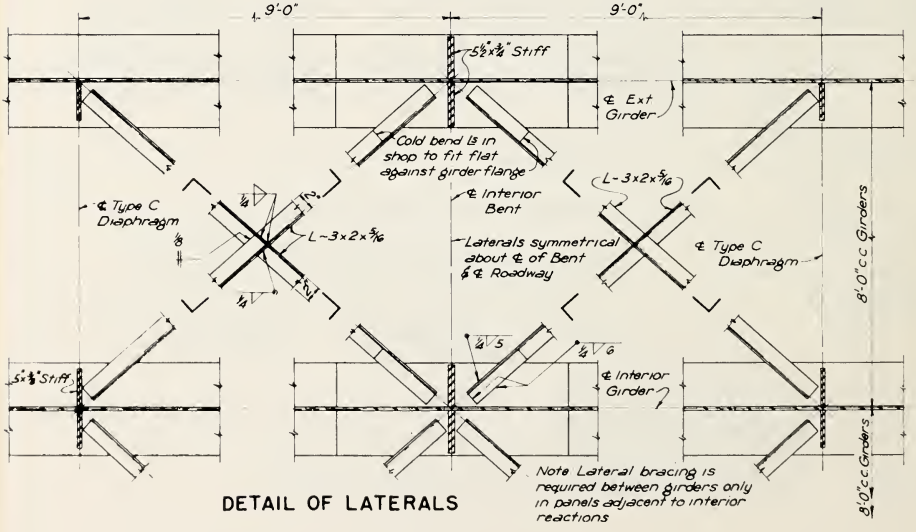
INTERIOR FIXED SHOE

Note Fixed Shoes are used at both Interior Piers



ELEVATION END VIEW

END EXPANSION SHOE


9'-0"
9'-0"
5½"x¾" Stiff
℄ Ext Girder
Cold bend is in shop to fit flat against girder flange
℄ Interior Bent
L-3x2x5/16
℄ Type C Diaphragm
Laterals symmetrical about ℄ of Bent & ℄ Roadway
℄ Type C Diaphragm
8'-0" c.c. Girders
5"x¾" Stiff
℄ Interior Girder
Note Lateral bracing is required between girders only in panels adjacent to interior reactions
8'-0" c.c. Girders

DETAIL OF LATERALS

DET

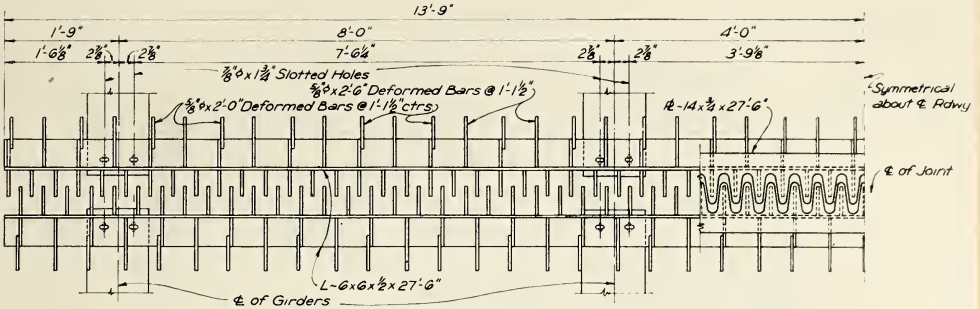
Note The single cut Grind off surface o

Note The assembled in the str checked, t The top s the roadw All Steel

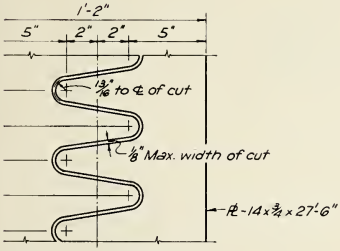
GENERA Girde in accord Design L

De Un Co

Im No

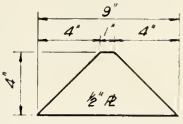


HALF PLAN OF FINGER JOINT

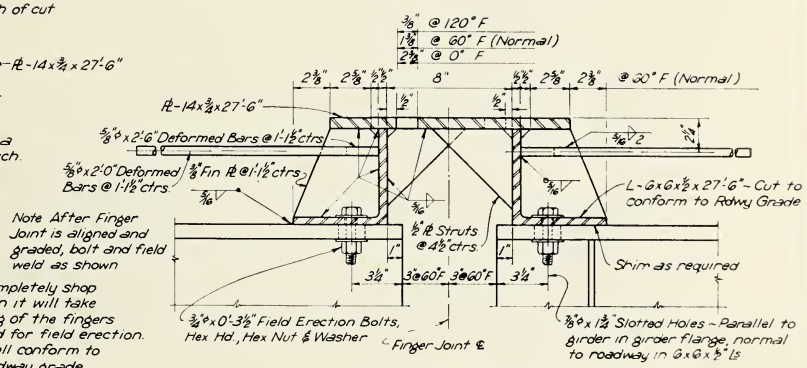


ETAIL OF SCROLL

The scroll shall be made by a cut of a machine guided torch. off burrs on the roadway e of the cut.



DETAIL OF STRUT



SECTION THROUGH FINGER JOINT

Note After Finger Joint is aligned and graded, bolt and field weld as shown

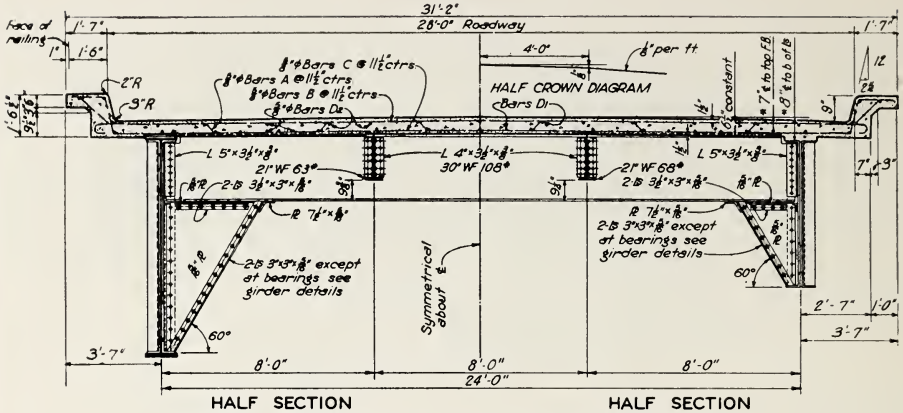
The Finger Joint shall be completely shop bled, adjusted to the position it will take structure, and the meshing of the fingers ed, fitted and match marked for field erection. op surface of the joint shall conform to oadway crown and the roadway grade. All metal for Finger Joints shall be Structural

RAL NOTES:
rders are designed for H 20-44 Loading ordance with A A S H O 1949 Specifications.
n Loads for the girders are as follows:

	Exterior Girder	Interior Girder
Dead Load	950 # p.l.f.	780 # p.l.f.
Uniform Live Load	400 # p.l.f.	512 # p.l.f.
Concentrated Live Load		
(for Moment)	11,250 #	14,400 #
(for Shear)	16,250 #	20,800 #

Impact in accordance with the Specifications
No camber is required for the girders.

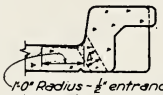
Figure 16



HALF SECTION
THROUGH INTERIOR SPAN

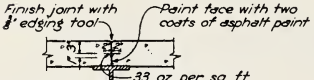
HALF SECTION
THROUGH END SPAN

* Add to these dimensions: dead load deflection due to concrete slab plus deflection of girders due to their own dead load



DETAIL OF DRAIN

4"×6" formed drain. Trowel entrance to give a ½" entrance depression. Provide a ½" drip bead around drain. Bend steel to clear drain 1". Place drains 10' from piers and 20'± c-c



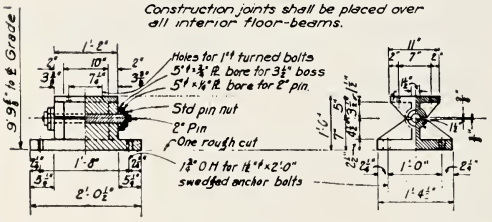
CONSTRUCTION JOINT

Construction joints shall be placed over all interior floor-beams.

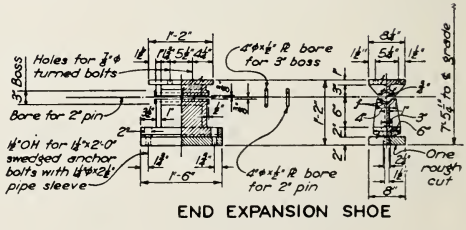
NOTE:- All materials for shoes shall be cast steel except base plates for end expansion shoes may be rolled steel. All re-entrant corners shall be filleted ½"

DEFLECTION FOR L.L. + I

	End Span	Int. Span
Calculated	0.94"	1.36"
Allowable	1.49"	1.95"



FIXED SHOE
(4 Required)


1'-2"
Holes for ⅞" ϕ turned bolts
4" ϕ × ½" PL bore for 3" boss
3" Boss
Bore for 2" pin
½" OH for 1¼" × 2'-0" swedged anchor bolts with ¼" × 2½" pipe sleeve
1'-6"
4" ϕ × ½" PL bore for 2" pin
One rough cut
7-5½" to ℄ grade
8"

END EXPANSION SHOE

Figure 17

Neither of these units have a participating floor and no comparisons are offered concerning the weight which might be saved by such a floor.

"The erection of the welded bridge would be considerably easier and cheaper than would the erection of the riveted one. The sections between field splices for the welded unit weigh from ¾ to ½ as much as corresponding sections for the riveted girder. This means lighter construction equipment and greater ease of handling. The shoes for Exhibit B are field welded to the bottom girder flanges allowing for greater freedom in placing of anchor bolts for shoes and also allowing for some incorrectness in lengths of girder sections due to fabrication errors or shrinkage from field splicing. The diaphrams or cross bracing also have some freedom as to fabrication length. In general, the correctness of fit on the welded girder unit is not so acute as it is on the riveted unit. A recommended erection procedure for the girders of Exhibit B would be to place the two sections over the piers first and anchor them by means of tension cables tied to the bents. The sections in the end spans could then be lifted in place and bolted or tack welded to the abutting ends of the sections over the piers. Finally, the center section could be raised and secured in place. The field splicing should then follow before the girders are welded to their shoes. Each girder should be erected in this sequence and then the diaphrams should be welded in place. Minor distortions in the lines of the girders could be compensated for by haunching of the slab on the top girder flange. The amount of field welding for this type of structure is exceptionally small, although competent, qualified welders under intelligent supervision are required to make the field splices in the girders. The field fillet welds require a lesser degree of skill.

"The saving in steel of 25.8 per cent for Exhibit B does not reflect the total amount saved. It may be seen from a study of the plans that the welded unit is designed so that the waste in steel is limited to a few minor blocks and beveled cuts in the bracing. The larger trapezoidal sections of web plate on either side of the interior bearings are intended to be cut from a plate 144 inches wide by 29 feet long with no waste in steel. The riveted unit on the other hand has considerable waste from the curved soffit, from the blocking of the ends of floorbeams and stringers and most important of all from the punching and reaming of holes for the rivets. The per cent saved on the mill order of material would be around 35 per cent to 40 per cent.

"Any figures for costs and maintenance of necessity contain many variables too numerous to mention here. The figures presented in the summary do fit the conditions of the last few months in the locality with which the designers are familiar. The cost and maintenance figures presented do not include any allowance for engineering planning or supervision. The increased cost of painting the riveted girder is due to the larger surface area of the riveted unit as well as to its greater roughness due to riveted heads and riveted connections. The overall cost saving of 34.6 per cent for the welded girder would be somewhat increased by a saving in the cost of piers. This saving however would be minor and no estimate is made of it in this discussion.

"One special feature of the welded design is its decreased resistance to transverse wind forces. The comparatively shallow, open-webbed sections present a diminished area for the wind force to act against. Thus, only a small amount of bottom lateral bracing adjacent to the interior reactions was deemed necessary to care for the design wind load stresses. Another feature of the welded unit is the longitudinal stiffeners used in the deeper sections over the piers. These angle stiffeners are used to reduce the required thickness of the web plate and are oriented to give the maximum moment of inertia about the face of the web. The riveted unit was built without such stiffeners and the structure was not altered to include them for this comparison."

The shoes, laterals, and the finger joints are detailed in Figure 16. Two half sections on the riveted span are shown in Figure 17. As can be seen, the floorbeams are 30 WF 108 sections and all stringers in the two interior lines are 21 WF sections. The end spans (100 ft.) are divided into 5 panels and the center span (130 ft.) is divided into 7 panels

Not including finger joints or shoes, Exhibit A has 334,900 lb. of steel and Exhibit B has 248,400 lb. The riveted steel includes 6,559 lb. for rivet heads—the welded steel includes 2,288 lb. for shop welds and 228 lb. for field welds. The welded construction saves 25.8 per cent of steel, and Messrs. Randall and Bundy estimated the saving in cost to be 34.6 per cent, based on riveted steel costing $0.17 per lb. and welded steel costing $0.15 per lb.

Horace O. Titus, Cheyenne, Wyoming, compared his Exhibit B with two others which he termed Exhibit A and Exhibit AX, respectively. All three are three span continuous girder bridges. The spans are 50 ft., 62 ft.-6 in., and 50 ft. (see Figure 18), and the girder spacing is 8 ft.-8 in. The 7 in. reinforced concrete slab

which includes ½ in. for a wearing surface is carried directly on the four lines of girders.

For Exhibits A and B, the built-up girders have webs 44 in. deep as shown in Figures 19 and 20, respectively. The 36 WF sections with cover plates of Figure 21 are used in Exhibit AX. Mr. Titus made these remarks about his designs.

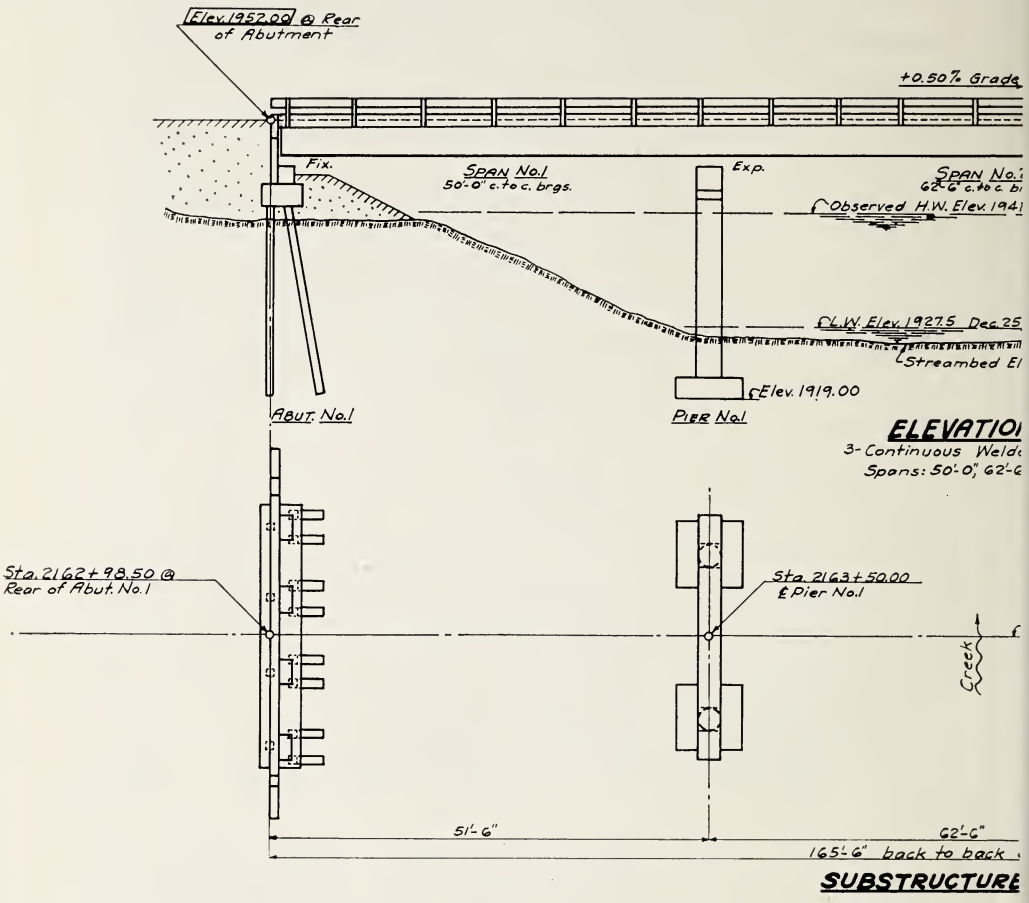
"The economy of the continuous girder bridge has, in recent years, given it an important place among contemporary highway bridges both large and small. This type of structure gives us a pleasing aesthetic deck structure besides its economy and utility. It requires fewer bearings for support compared with some other types of structures and also requires fewer contraction and expansion joints across the roadway, thereby producing a smoother riding surface. Also, because of fewer joints, it provides a tighter deck for the disposal of surface drainage which may produce unsightly stains after seeping through a joint or create maintenance problems because of alternate freezing and thawing in the joint.

"Many state highway departments in the United States have prepared standard design drawings for this type of structure, the personnel of highway bridge departments are all familiar with this type of design, and a very large number of continuous span girder bridges have been constructed in the last few years. Therefore, because of its utility and popularity, the continuous girder type of bridge was selected for the Exhibits presented with this paper. A relatively small bridge was also chosen to show the saving possible in cost and steel in the small bridge which makes up the largest part of the highway bridge construction. A larger bridge could be expected to produce a proportionately greater saving.

"A slightly greater field than this contest requires was investigated to work up the exhibits presented. This field covered various kinds of three span continuous bridges of the same span lengths and ratios as those given in the exhibits. Although only two exhibits are required by this contest, a third exhibit entitled 'Exhibit AX' is included. Exhibit AX shows a three span bridge with span ratios of 1 to 1.25 and with variable moment of inertia, this bridge to be constructed of riveted wide-flange girders. Exhibit A shows the same bridge using riveted plate girders. Exhibit B presents the same structure using welded plate girders.

"In addition to these three exhibits, investigations were made on welded wide-flange construction, welded split-tee construction, and continuous welded plate girders with participating concrete slab.

Note: Elevations shown thr
top of roadway slab



DESIGN DATA

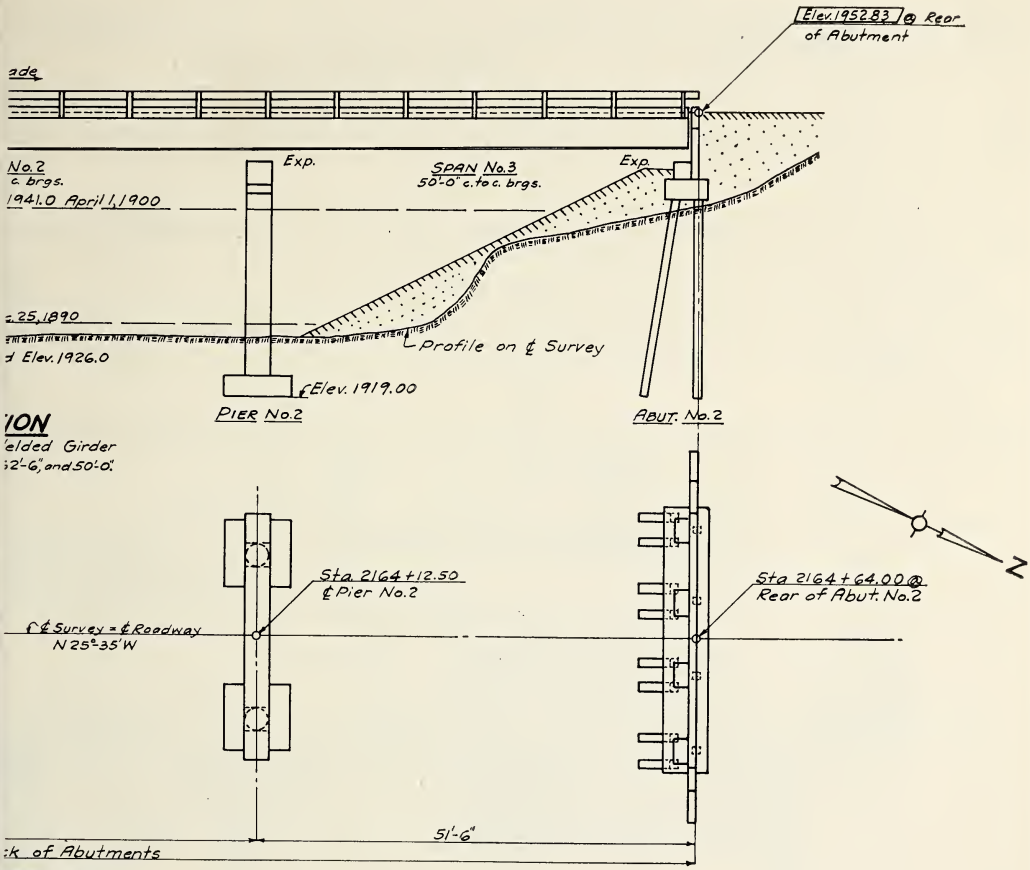
SPECIFICATIONS: A.A.S.H.O. Specifications, 1949 edition, and A.W.S. 1947 edition of "Standard Specifications for Welded Highway and Railroad Bridges."

LIVE LOAD: H 20-S16-44 on two lanes.

ROADWAY WIDTH: 30'-0" clear between curbs.

STRUCTURAL STEEL: A.S.T.M. A7-46, estimated per A.I.S.C. Code of Standard Practice adopted 1924, rev. Dec. 1, 1946.

, thus: Elev.1952.00 indicate
lab on ℄ of Roadway.



INDEX OF SHEETS

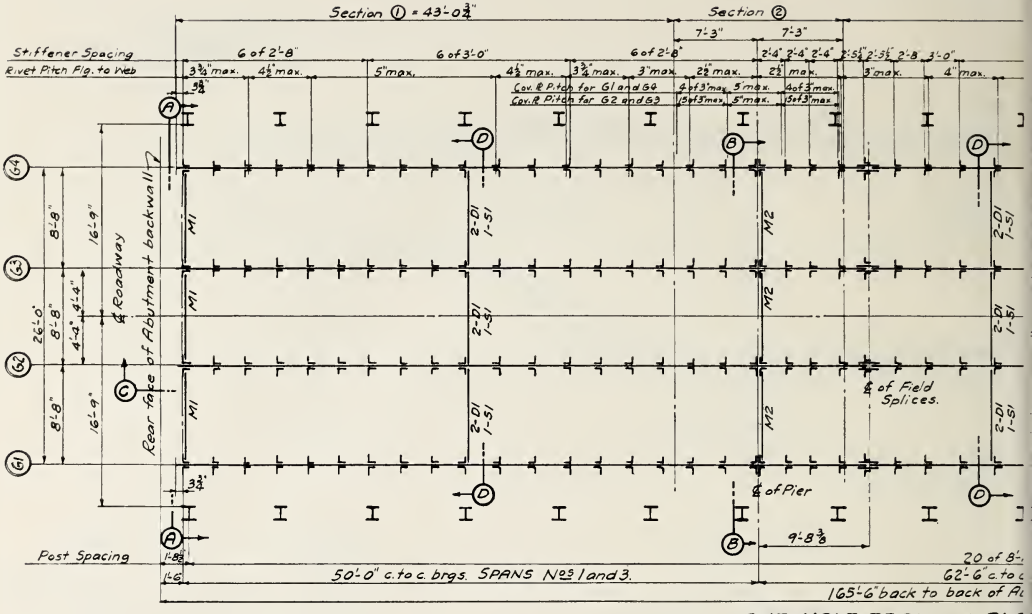
General Drawing_____Sheet No.1.
Superstructure Details___Sheets Nos. 2 and 3.
Structural Steel Estimate____Sheet No.4.
Design Computations___Sheets Nos.5,6, and 7.

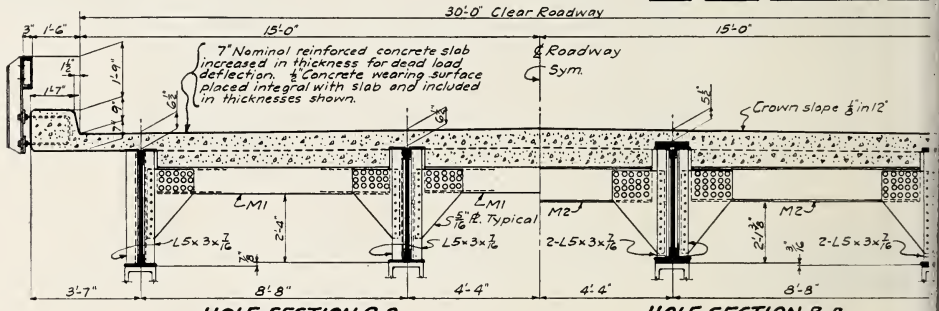
SUPERSTRUCTURE TOTAL ESTIMATED QUANTITIES

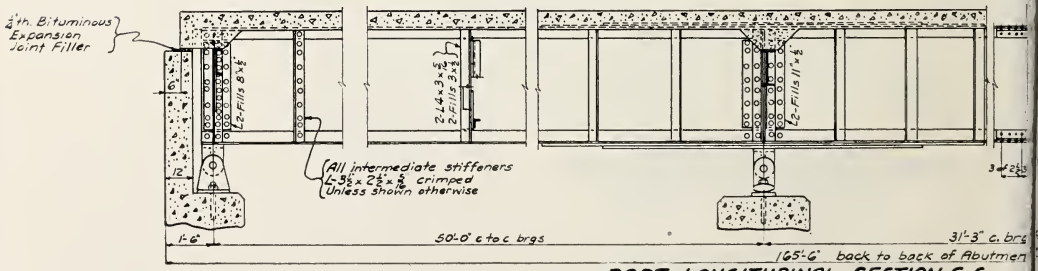
Structural Steel① ____ 96807 lb.
Concrete (3000 p.s.i)② ____ 135.5 cu.yd
Reinforcing Steel ___ Not estimated.
Floor Drains_____" "

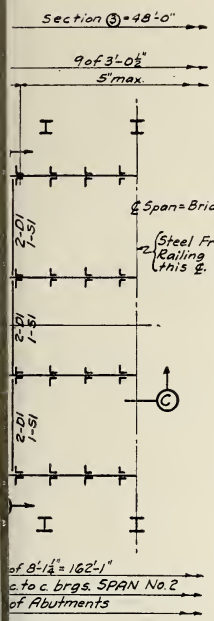
① See Sheet No.4
② {Weight/lin.ft. of bridge = 3316 lb.
{Weight/lin.ft. over girders = 3244 lb.
Total Weight/lin.ft. of Concrete and Structural Steel (exclusive of bearings and expansion unit) for length of steel part of superstructure = 3800 lb.

Figure 18

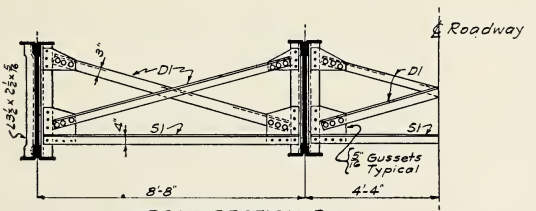

Section ① = 43'-0 3/4"
Section ②
Stiffener Spacing
Rivet Pitch Flg. to Web
6 of 2'-8"
6 of 3'-0"
6 of 2'-8"
7'-3"
7'-3"
2'-6" 2'-6" 2'-6" 2'-5½" 2'-5½" 2'-8" 3'-0"
3 3/4" max. 4½" max. 5" max. 4½" max. 3 3/4" max. 3" max. 2½" max. 2½" max. 3" max. 4" max.
Cov. & Pitch for G1 and G6
Cov. & Pitch for G2 and G5
4 of 3" max. 5" max. 6 of 3" max.
5 of 3" max. 5" max. 5 of 3" max.
Rear face of Abutment backwall
℄ Roadway
℄ of Field Splices.
℄ of Pier
M1
M2
2-D1 1-S1
Post Spacing
50'-0" c. to c. brgs. SPANS Nos 1 and 3.
165'-6" back to back of Ab
ONE HALF FRAMING PLA


30'-0" Clear Roadway
15'-0"
15'-0"
℄ Roadway Sym.
7" Nominal reinforced concrete slab increased in thickness for dead load deflection. ½" Concrete wearing surface placed integral with slab and included in thicknesses shown.
Crown slope ⅜" in 12"
M1
M2
L5 x 3 x 7/16
2-L5 x 3 x 7/16
5/16" R. Typical
3'-7"
8'-8"
4'-4"
4'-4"
8'-8"
HALF SECTION A-A
HALF SECTION B-B

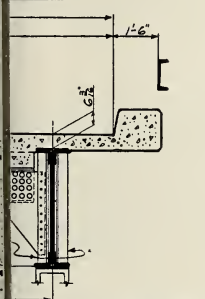

¼th. Bituminous Expansion Joint Filler
2-Fills 8 x ½
2-L4 x 3 x 5/16
2-Fills 3 x ½
2-Fills 11 x ½
All intermediate stiffeners L 3½ x 2½ x 7/16 crimped Unless shown otherwise
50'-0" c to c brgs
165'-6" back to back of Abutmen
PART LONGITUDINAL SECTION C-C
(Typical)

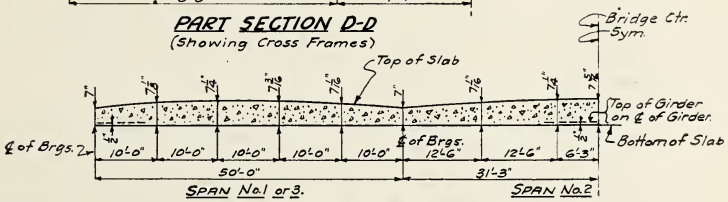


STRUCTURAL STEEL SCHEDULE				
GIRDERS			SLAB SUPPORTS AND CROSS FRAMES	
SECTION	G1 and G4	G2 and G3	MARK	DESCRIPTION
1 and 3	3'-8½" 4-L6x4x½ 1-Web 44x5/16	3'-8½" 4-L6x4x½ 1-Web 44x5/16	M1	10 C 15.3
			M2	14 WF 30
2	3'-8¼" 4-L6x4x½ 1-Web 44x5/16 Cov. PL 12x5/16x14'-6 Cov. PL 12x5/16x14'-6	3'-8¼" 4-L6x4x½ 1-Web 44x5/16 Cov. PL 12x½x10'-6 Cov. PL 12x5/8x14'-6 Cov. PL 12x½x14'-6 Cov. PL 12x½x10'-6	D1	L 3 x 2½ x 5/16
			S1	L 4 x 3½ x 5/16

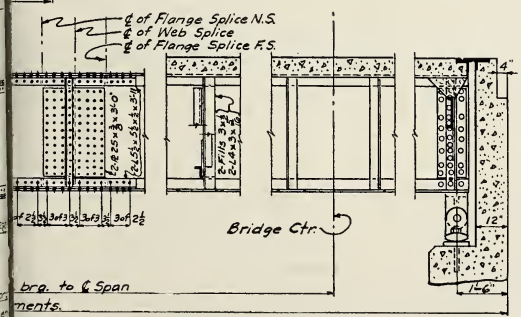


PART SECTION D-D
(Showing Cross Frames)



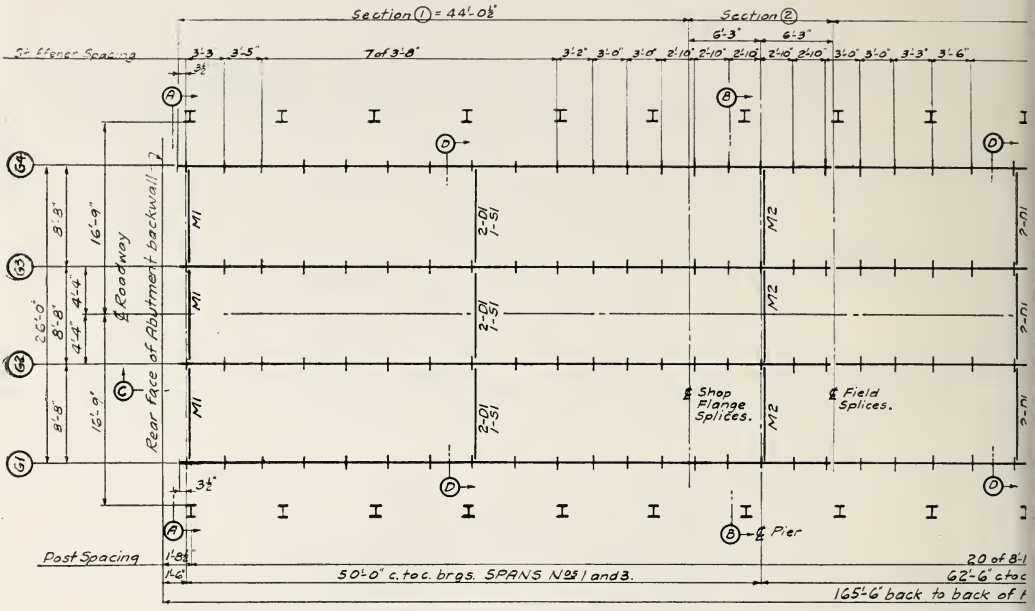


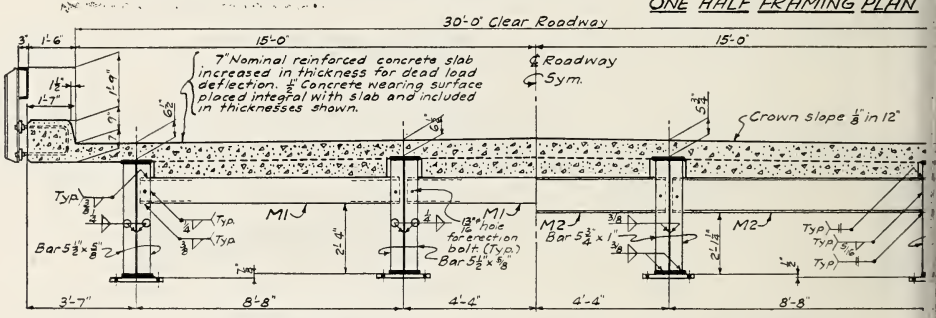
SLAB THICKENING DIAGRAM FOR DEAD LOAD DEFLECTION

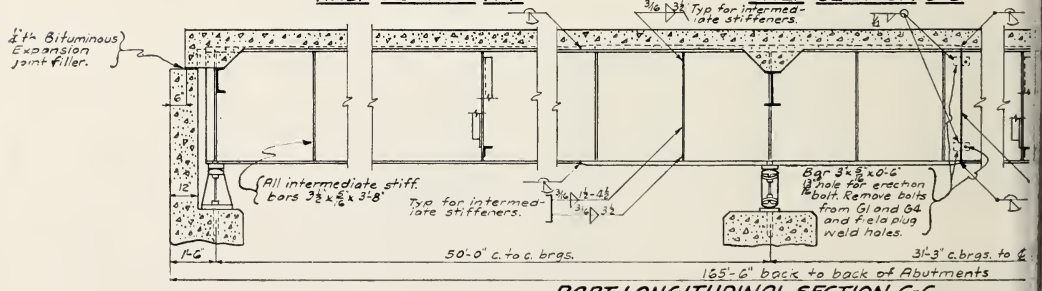


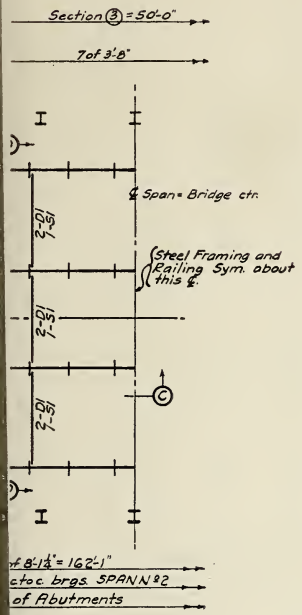
Rivets: ¾" Bh.

Figure 19

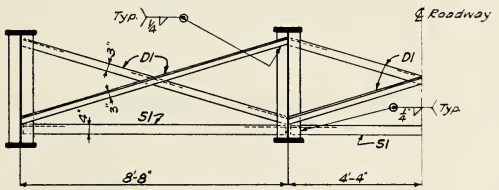

Section ① = 44'-0½"
Section ②
Ctr Effect Spacing
7 at 3'-8"
Rear Face of Abutment backwall
℄ Roadway
M1
M2
2-D1 1-S1
℄ Shop Flange Splices.
℄ Field Splices.
℄ Pier
Post Spacing
50'-0" c. to c. brgs. SPANS Nos 1 and 3.
165'-6" back to back of
ONE HALF FRAMING PLAN


30'-0" Clear Roadway
15'-0"
15'-0"
7" Nominal reinforced concrete slab increased in thickness for dead load deflection. 2" Concrete wearing surface placed integral with slab and included in thicknesses shown.
℄ Roadway
Sym.
Crown slope 1" in 12"
M1
M2
Bar 5½" x 5" x ⅝
Bar 5½" x ⅝
Bar 5¾" x 1"
13/16" hole for erection bolt (Typ.)
3'-7"
8'-8"
4'-4"
4'-4"
8'-8"
HALF SECTION A-A
HALF SECTION B-B

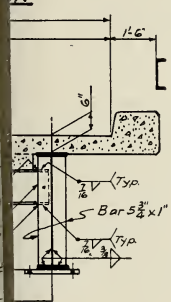

(½" Thk Bituminous) Expansion joint filler.
Typ for intermediate stiffeners.
All intermediate stiff. bars 3½ x ⅝ x 3'-8"
Typ for intermediate stiffeners.
Bar 3" x ⅝ x 0'-6"
13/16" hole for erection bolt. Remove bolts from G1 and G4 and field plug weld holes.
50'-0" c. to c. brgs.
31'-3" c. brgs. to ℄
165'-6" back to back of Abutments
PART LONGITUDINAL SECTION C-C
(Typical)

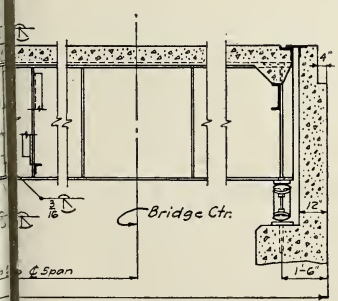


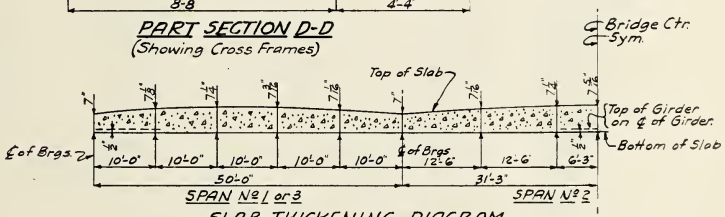
STRUCTURAL STEEL SCHEDULE				
GIRDERS			SLAB SUPPORTS AND CROSS FRAMES	
SECTION	G1 and G4	G2 and G3	MARK	DESCRIPTION
1 and 3	Flg. 12"×½" / 3'-9" / 3/16 Web 44"×5/16" / 3/16 / Flg. 12"×½"	Flg. 12"×5/8" / 3'-9¼" / ¼ Web 44"×5/16" / ¼ / Flg. 12"×5/8"	M1	10 C 15.3
			M2	14 WF 30
2	Flg. 12"×1" / 3'-10" / 5/16 Web 44"×5/16" / 5/16 / Flg. 12"×1"	Flg. 12"×1⅜" / 3'-10⅜" / ⅜ Web 44"×5/16" / ⅜ / Flg. 12"×1⅜"	D1	L 3"×2½"×5/16"
			S1	L 4"×3½"×5/16"



PART SECTION D-D
(Showing Cross Frames)





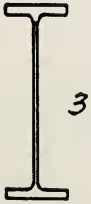


SLAB THICKENING DIAGRAM FOR DEAD LOAD DEFLECTION

Figure 20

Figure 21

STRUCTURAL STEEL SCHEDULE

GIRDERS			SLAB SUPPORTS AND CROSS FRAMES	
SECTION	G1 and G4	G2 and G3	MARK	DESCRIPTION
1 and 3	36WF150	36WF160	M1	10C15.3
			M2	14WF30
2	 36WF150	Cov. Pl 12 x $\frac{3}{4}$ x 10'-6" Cov. Pl 12 x $\frac{3}{4}$ x 14'-6" 36WF160 Cov. Pl 12 x $\frac{3}{4}$ x 14'-6" Cov. Pl 12 x $\frac{3}{4}$ x 10'-6"	D1	L 3 x 2$\frac{1}{2}$ x $\frac{5}{16}$
			S1	L 4 x 3$\frac{1}{2}$ x $\frac{5}{16}$

No exhibits were prepared for the last three named types. It is obvious after reviewing Exhibit AX and Exhibit B, that the welded wide-flange girders (similar to Exhibit AX except using welding) would require more steel than Exhibit B, and computations showed that Exhibit B would require less steel than either the welded split-tee construction or the continuous composite type of construction with participating slab.

"The exhibits herewith presented do not consider the condition of restricted headroom as many bridges over small streams do not present this problem. These exhibits, however, would be used at locations not controlled by clearance and for conditions in which economical depths could be used for the girders. Exhibit AX was therefore included to show the wasteful use of rolled sections at places where welded built-up girders could have served and would have been more economical, saving both steel and money. It should be pointed out, that in some instances it will be cheaper to use girders of economical depth and to provide for a raise in grade than to use wide-flange girders of shallower and uneconomical depth since the cost of earthwork in many cases is cheaper than the cost of structures.

"As the time and labor expended in the design of a continuous structure with variable moment of inertia, such as presented in these exhibits, is usually considerable compared to the design of simpler structures, a set of unpublished influence lines covering moments, shears, and deflections was used. These influence lines, prepared a few years ago, reduce the labor and time involved to the extent that the time required in designing a structure of this type is about the same as that required to design simple spans.

"One of the known steel saving features of the continuous span with variable moment of inertia, of course, is that a small haunch or a small increase in moment of inertia of that part of the girder over the interior supports can absorb much moment and also decrease the midspan deflection. Exhibits A, AX, and B all show that a small amount of steel used over the interior supports can produce a considerable saving of steel in the superstructure of a bridge of this type.

"Please refer to Exhibit B and note that practically all the material used is of small thicknesses and therefore can be of A.S.T.M. A-7 steel and presents no difficulties or unusual complications in welding. This type of work can be fabricated by any shop which has only a small investment in equipment as well as by any of the fully equipped large shops."

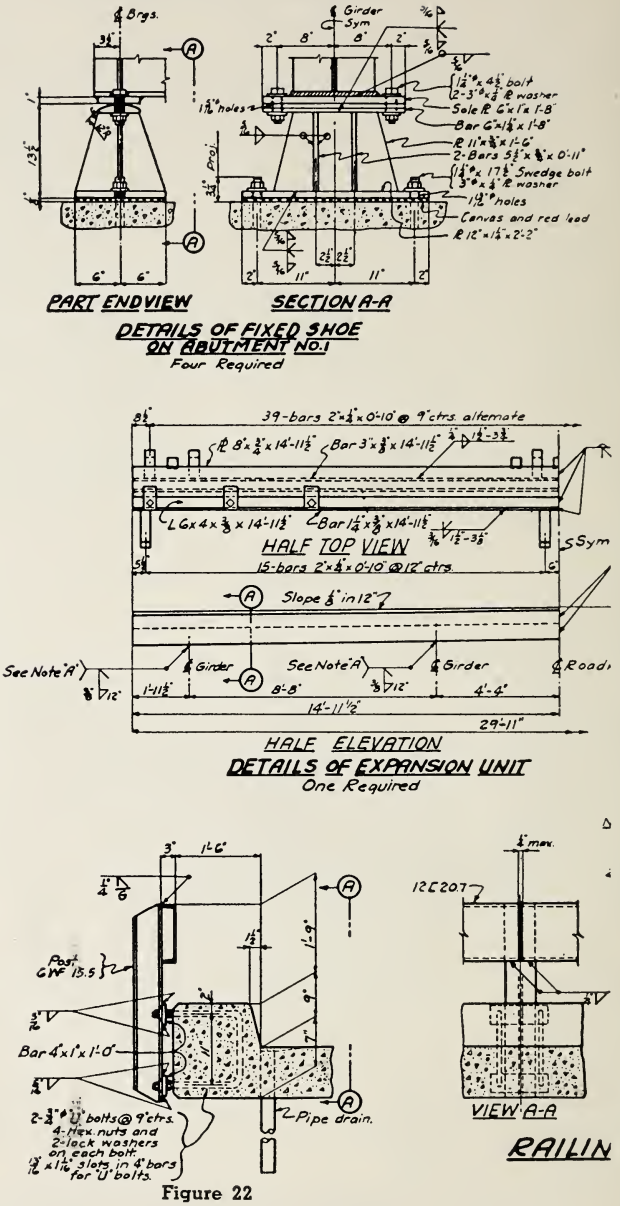


Figure 22

The total steel weight including shoes, rockers, and railing are 128,012 lb. for Exhibit A, 141, 289 lb. for AX, and 96,807 lb. for B. The estimated costs for the erected structures in cents per lb. are 20.68, 17.28, and 16.27, respectively. Although Exhibit AX weighs more than A, its estimated cost is less; however, the cost

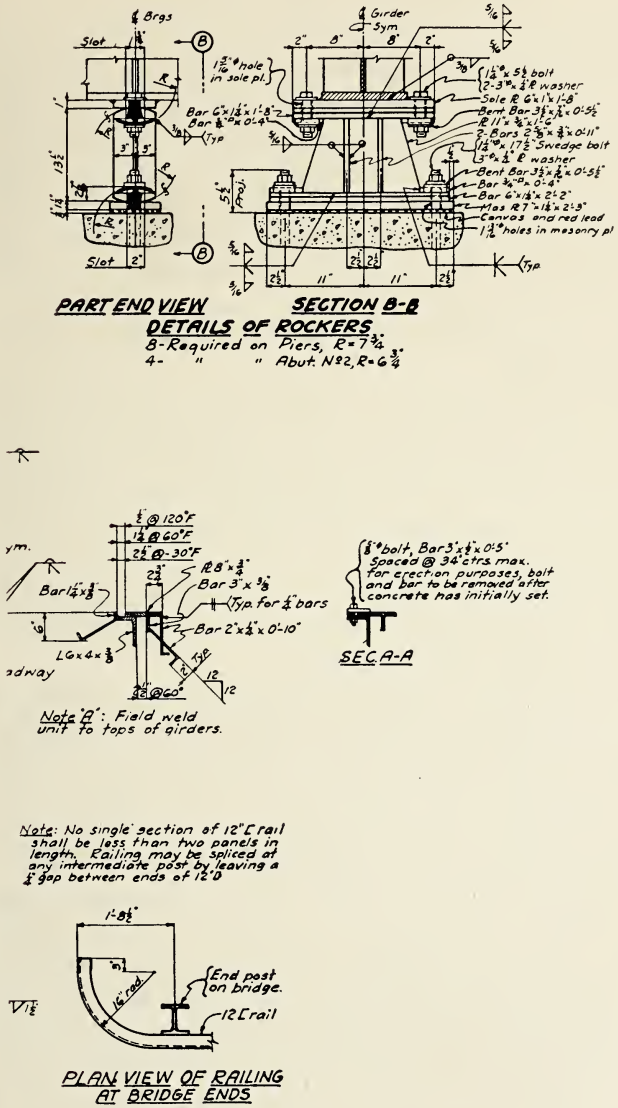

Brgs
Slot
Girder
Sym.
hole in sole pl.
Bar 6"x1½"x1'-8"
Bar ⅜"x 0'-4"
Typ
Sole R 6"x1"x1'-8"
Bent Bar 3½"x¾"x 0'-5½"
R 11"x ⅜"x 1'-6"
2-Bars 2"x⅜"x ⅜"x 0'-11"
Swedge bolt
R washer
Bent Bar 3½"x¾"x 0'-5½"
Bar ¾"ø x 0'-4"
Bar 6"x⅜"x 2'-2"
Plate R 7"x⅜"x 2'-3"
Canvas and red lead
holes in masonry pl
Proj.
Typ
PART END VIEW
SECTION B-B
DETAILS OF ROCKERS
8- Required on Piers, R= 7¾"
4- " " Abut. N°2, R= 6¾"
@ 120°F
@ 60°F
@ 30°F
R 8"x¾"
Bar 3"x⅝"
Typ. for ¾" bars
Bar 2"x¾"x 0'-10"
Bar 1¼"x⅜"
LG x 4"x⅜"
Note "A": Field weld unit to tops of girders.
⅞" bolt, Bar 3"x¼"x 0'-5" Spaced @ 34" ctrs. max. for erection purposes, bolt and bar to be removed after concrete has initially set.
SEC. A-A
Note: No single section of 12" C rail shall be less than two panels in length. Railing may be spliced at any intermediate post by leaving a ¼" gap between ends of 12'0
End post on bridge.
12 C rail
PLAN VIEW OF RAILING AT BRIDGE ENDS
NG DETAILS

Figure 22

savings of B over AX is 35 per cent and the weight savings of B over A is 24 per cent. Exhibit B has a total of 1,256 lb. of shop welds and only 80 lb. of field welds. Of the total weight of 96,807 lb., the railing accounts for 10,727 lb. and the shoes and rockers for 5045 lb. Details of these items are shown in Figure 22.

Continuous Girder Bridges With Floorbeams

M. O. Elkow, Yonkers, New York, used stringers and floorbeams to support the 5½ in. reinforced concrete roadway of his three span continuous bridge. The total length of 520 ft. is divided into a center span of 200 ft. and two end spans of 160 ft. each. The girders have a constant depth of 9 ft. and are spaced 42 ft. apart. The web thickness is 7/16 in. except for the region near the center piers where the thickness is ½ in. For the riveted girders, the flanges consist of 8 x 8 x 1⅛ angles with 24 in. cover plates of varying thickness. The welded girders have flange plates 2½ in. thick with varying width.

The floorbeams frame between the girders and support the outside stringer and fascia beam on each side with tapered ends that cantilever beyond the girders. Figure 23 is a transverse section of the four lane bridge with its two sidewalks that shows a welded floorbeam. The 10 ft. spacing for the floorbeams and the 6 ft. spacing for the stringers (16 WF 36) are the same for both the welded and riveted bridges. Figure 24 is a framing plan. The riveted floorbeams are 33 WF 200—the welded are 36 WF 194.

Mr. Elkow discussed his design as follows:

"The structure selected for entry is not unusual in its idea. It is suited for any number of typical installations requiring the bridging of highway traffic over valleys, railroads, rivers or other highways. It is adaptable to so many of the cases arising in the design of new toll highways and throughways for which there is a considerable interest and demand during the present time.

"The shape of the structure is simple without haunches or any sort of arching. Higher strength steel is used only for the flanges over the supports where the stresses are much higher within a relatively short length of the structure. For longer spans it may be preferable to use higher strength steel throughout for the main material. However, for this design this did not appear to be necessary.

"Roadway lanes were made wider than the minimum required by the Standard Specifications for Highway Bridges of the American Association of State Highway Officials. It was felt that today's faster moving traffic requires lanes thirteen feet wide. The surface of asphalt was another feature considered necessary for fast moving traffic. Concrete surfaces are never sufficiently smooth regardless of the amount of care which may have been taken. The slight waves in the surface cannot apparently be avoided. For slow

moving traffic this is not noticed. But for fast moving traffic these waves act as a defective pavement in jarring the vehicles. Asphalt surfaces can be better controlled and replaced more easily.

"Considerable study was made of a participating floor during the course of design for the entry. However, it was decided that a thin one-way concrete slab on continuous stringers was more economical of steel and had the further advantage of being less difficult and more economical to construct.

"Use of a constant thickness of two and one-half inch plates of varying width for the flanges of the welded plate girders was decided after considerable thought. Stitching of individual thinner plates is customarily used for a riveted flange design. Varying the width was simpler and resulted in a much better girder section. The only disadvantage in the use of the thick flange plates is that a one-half inch fillet weld is required for the connection between the flanges and the web plates. Actual stresses do not require such a large weld. However, because of the thickness of these plates, the American Welding Society in the Standard Specifications for Welded Highway and Railway Bridges, requires larger welds.

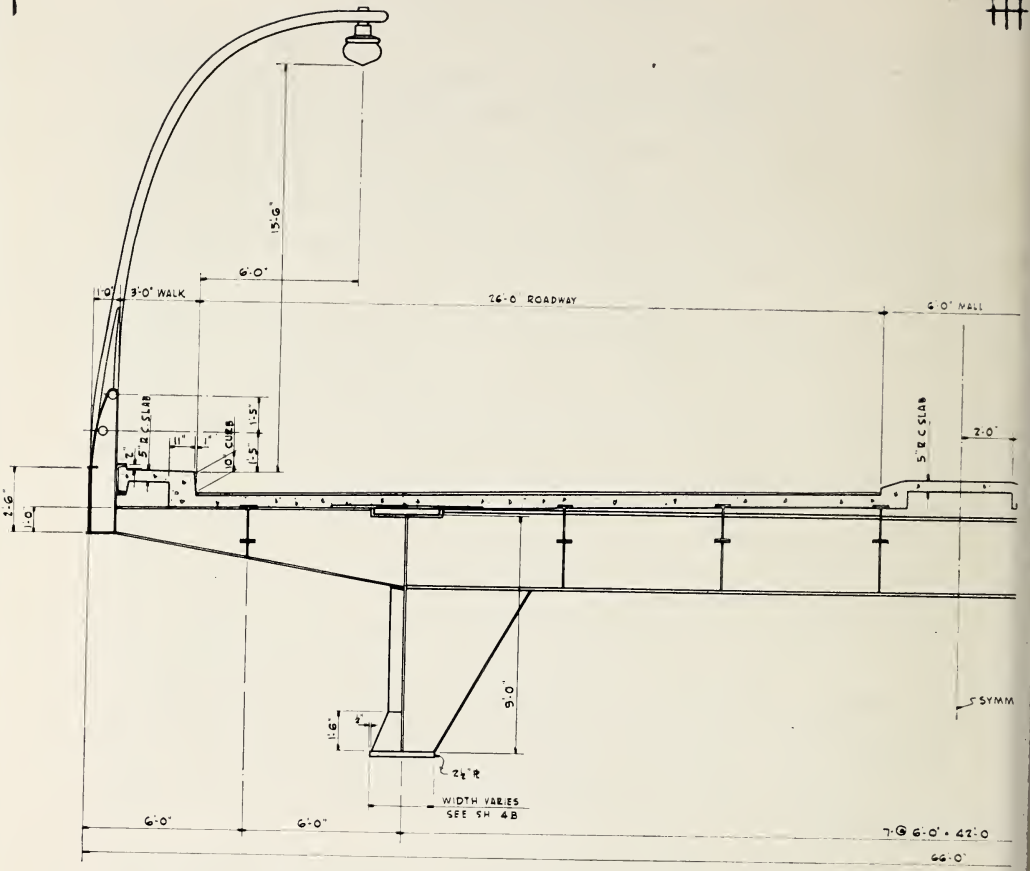
"Constant thickness of flange plates also has a considerable advantage in simplifying framing of floorbeams and cantilever beams. Variation in width of these plates does not affect the details of fabrication especially in the case of welded construction. A varying width flange on the other hand, is not practical for riveted structures."

Details for the welded girders and a longitudinal section are shown in Figure 25. For the riveted girder, these same items are shown in Figure 26.

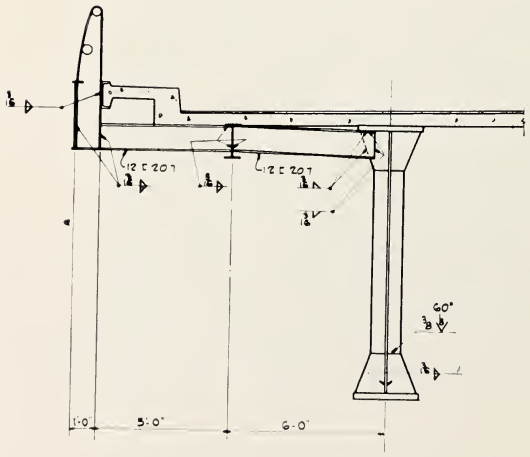
The quantities and costs as summarized by Mr. Elkow are:

"The two exhibits for the same design offer a considerable amount of interesting comparisons. These could not possibly be obtained by means other than a complete identical design using each method of construction. Quantities used in the following considerations are based on actual quantities as indicated in the details. No overruns for weight or painting have been included. Weights of bearing are not included. However, since both estimates are based on the same premises and are further compensated by an adjustment in unit prices, it is believed that the values obtained, furnish a valid comparison.

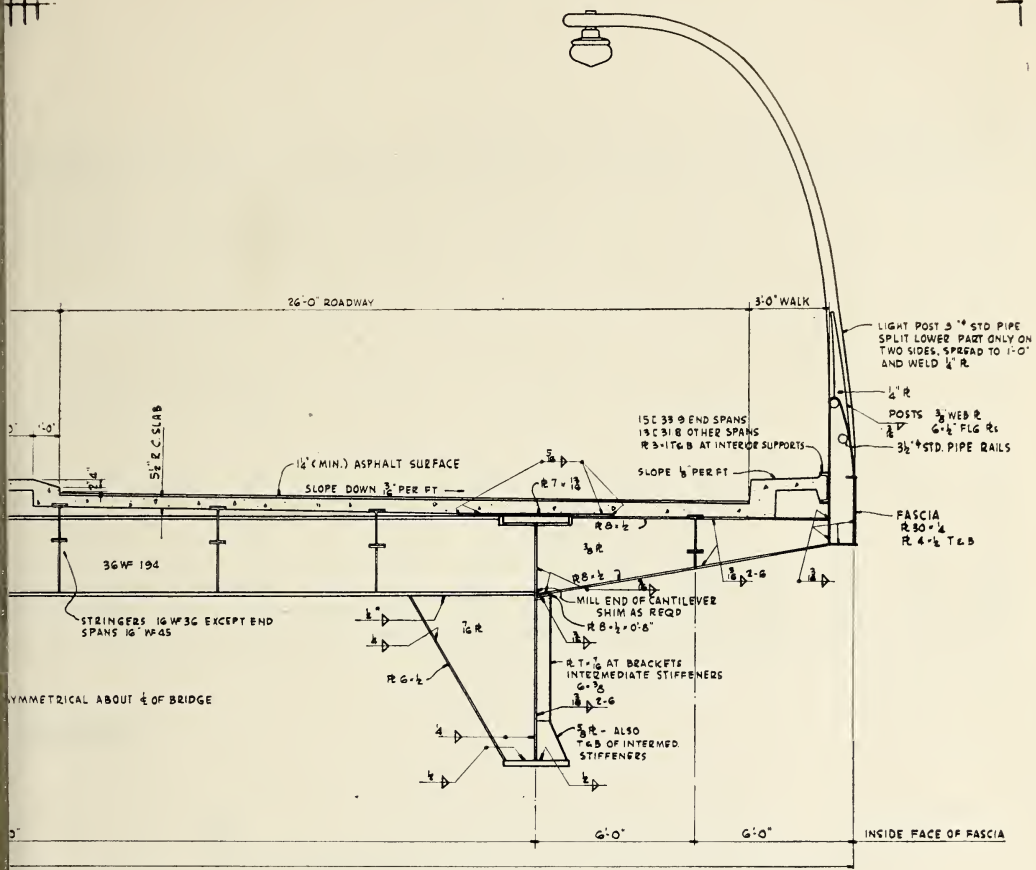
"Estimate for riveted design included under Exhibit A is as follows:



TYPICAL TRANSVE

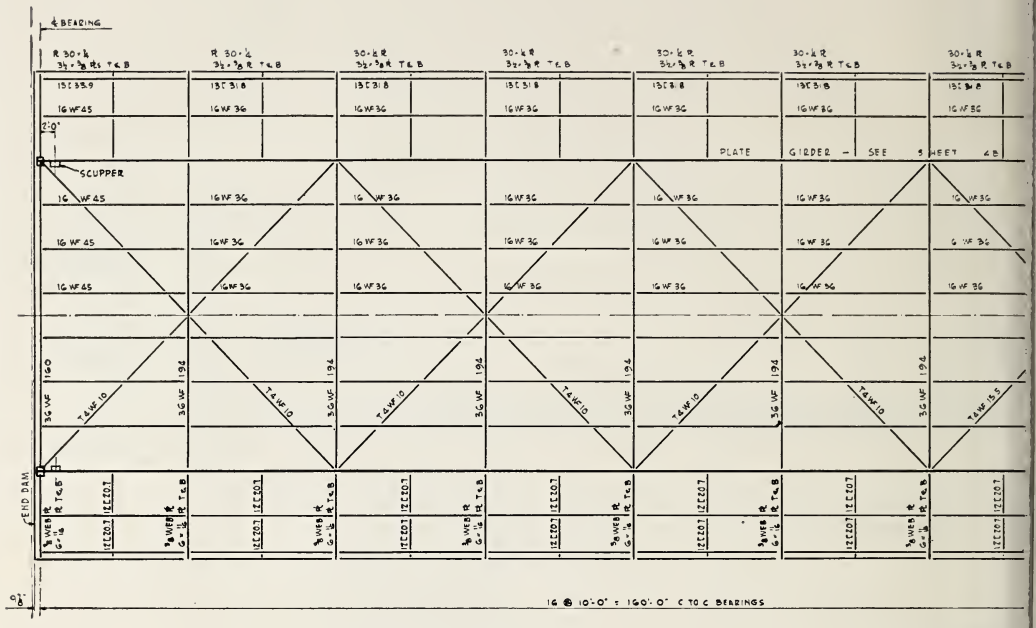


TRANSVERSE SECTION AT INTERMEDIATE BEAMS

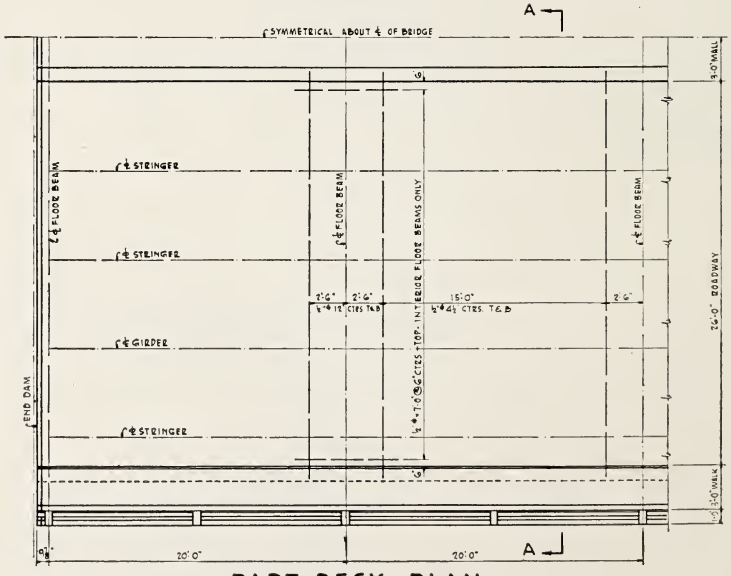

26'-0" ROADWAY
3'-0" WALK
LIGHT POST ∃ 4" STD PIPE SPLIT LOWER PART ONLY ON TWO SIDES, SPREAD TO 1'-0" AND WELD ¼" PL
¼" PL
POSTS ⅜" WEB PL 6"×½" FLG PLS
3½" STD. PIPE RAILS
15 C 33.9 END SPANS
13 C 31.8 OTHER SPANS
PL 3"×1T&B AT INTERIOR SUPPORTS
SLOPE ⅛" PER FT
FASCIA PL 30"×¼" PL 4"×½" T&B
5½" R.C. SLAB
1½" (MIN.) ASPHALT SURFACE
SLOPE DOWN 3/16" PER FT
36WF 194
STRINGERS 16WF36 EXCEPT END SPANS 16" WF 45
MILL END OF CANTILEVER SHIM AS REQD
PL 8"×½"×0'-8"
PL T=⅞" AT BRACKETS INTERMEDIATE STIFFENERS 6"×⅜"
⅝" PL - ALSO T&B OF INTERMED. STIFFENERS
YMMETRICAL ABOUT ℄ OF BRIDGE
6'-0"
6'-0"
INSIDE FACE OF FASCIA

ERSE SECTION

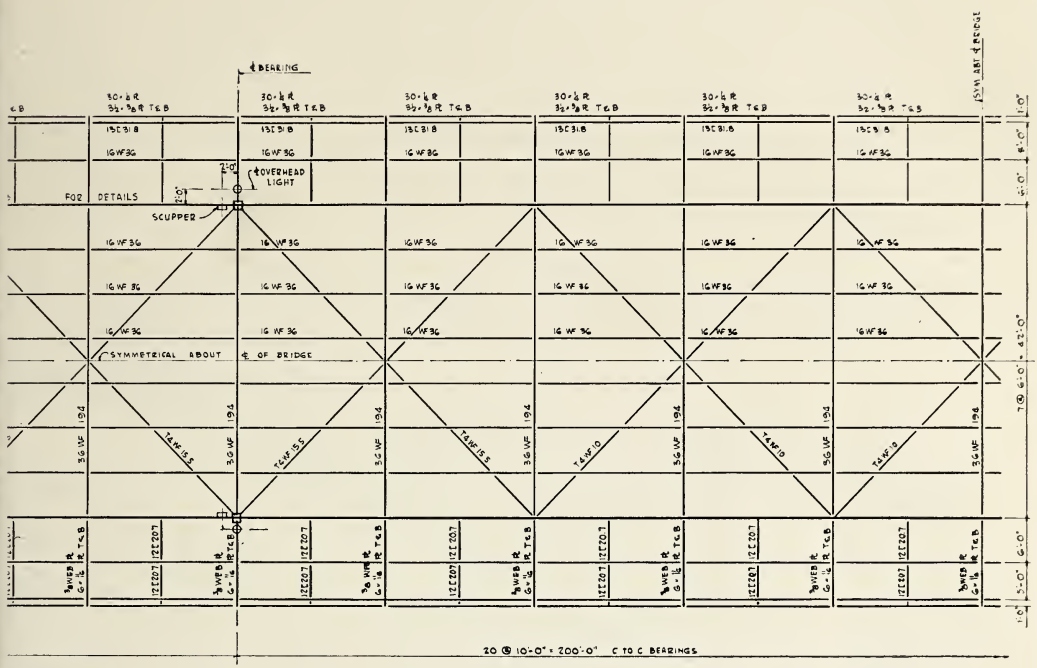
Figure 23



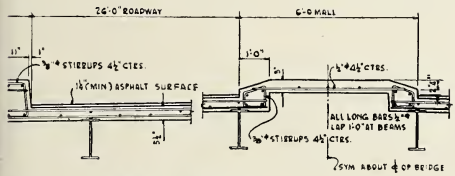
FRAMING



PART DECK PLAN

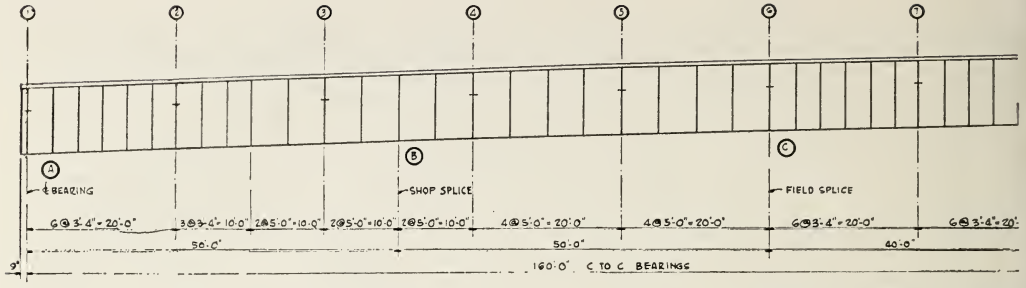


G PLAN

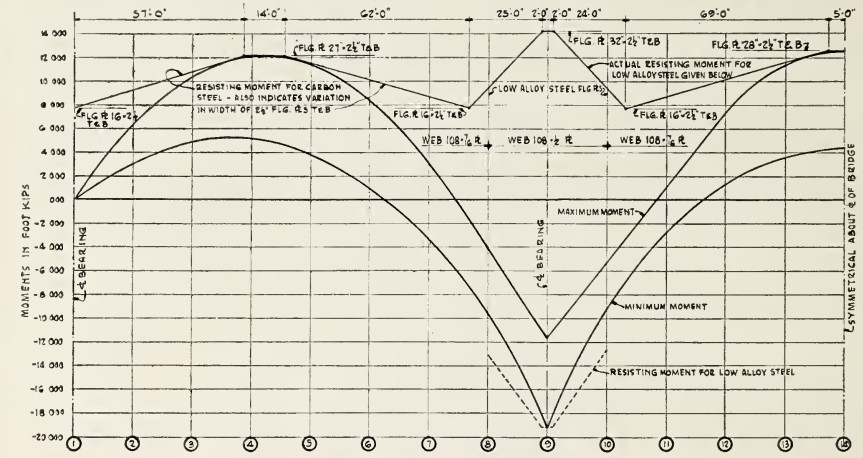


SECTION A-A

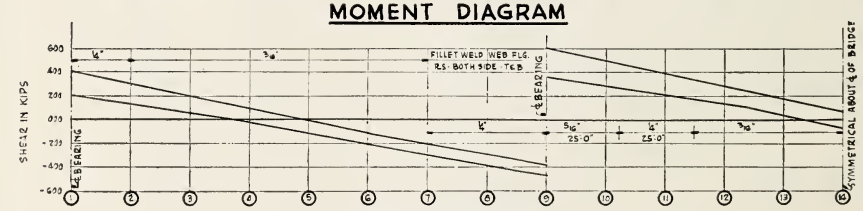
Figure 24



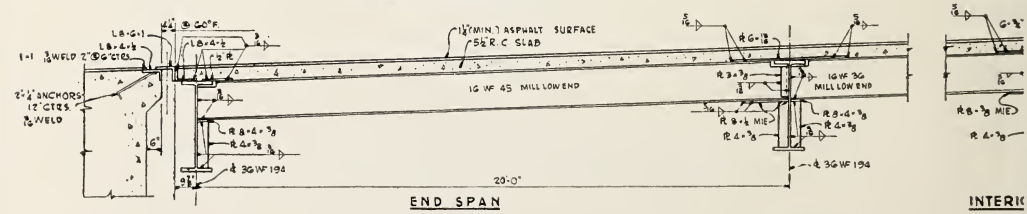
LONGITUDINAL SEC



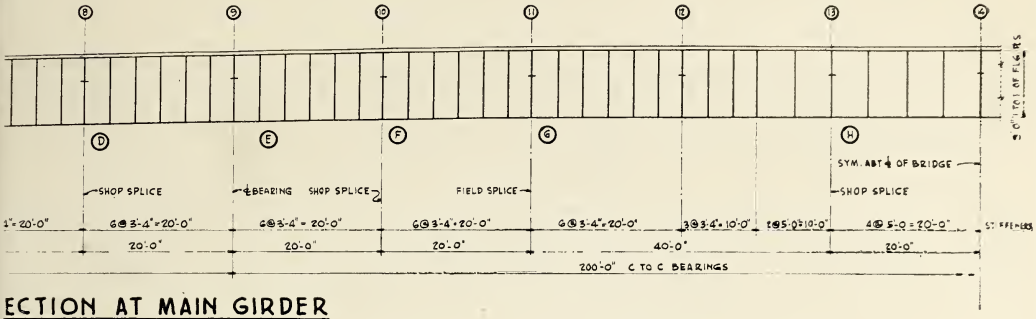
MOMENT DIAGRAM



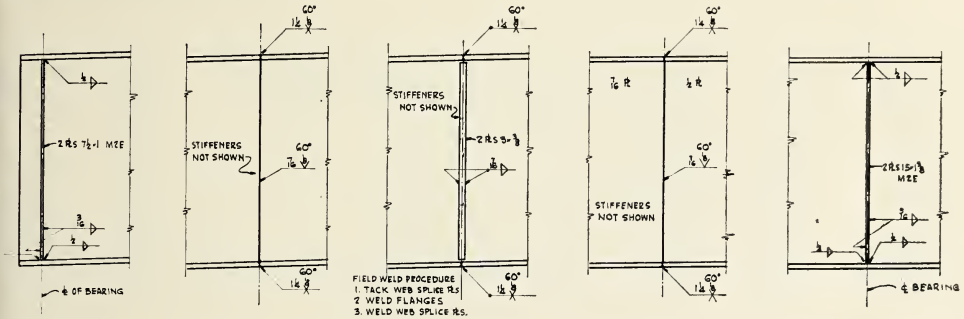
SHEAR DIAGRAM



LONGITUDINAL SECTION AT STRINGERS

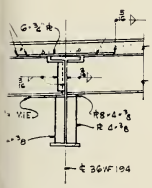


ECTION AT MAIN GIRDER



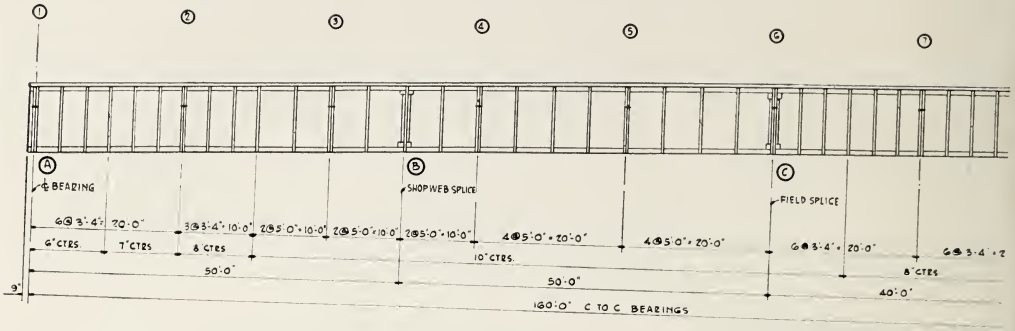
BEARING AT A | SHOP SPLICE AT B&H | FIELD SPLICE AT C&G | SHOP SPLICE AT D&F | BEARING AT E

GIRDER DETAILS

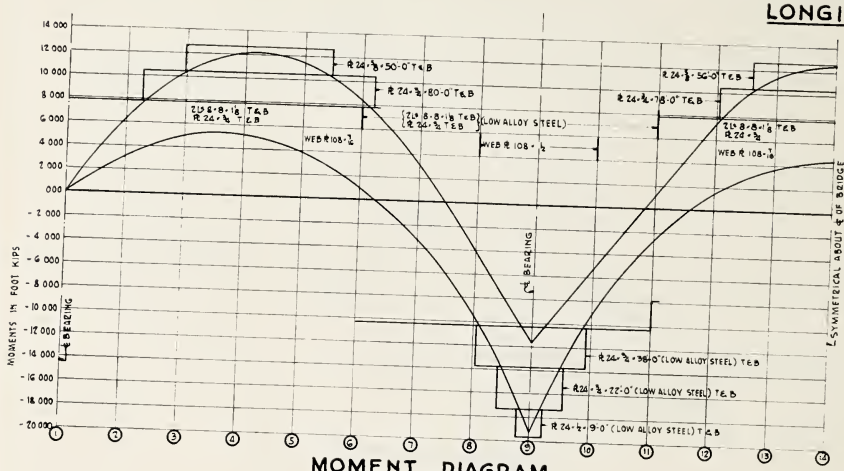


TERIOR SPANS

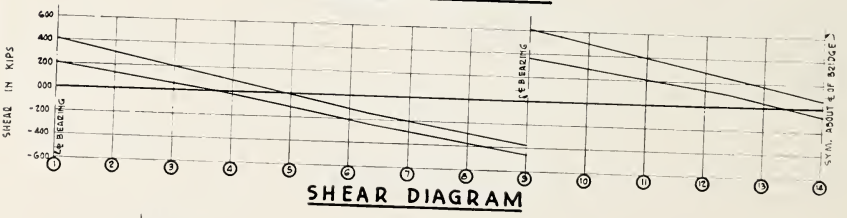
Figure 25



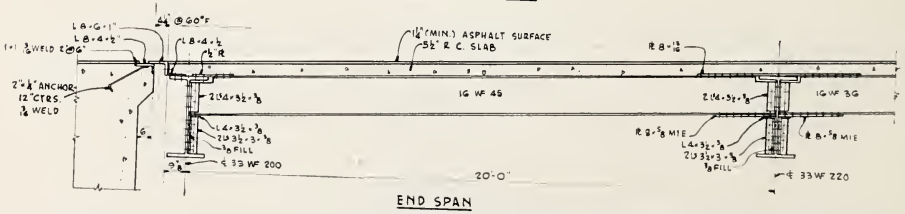
LONGITUDINAL SEC



MOMENT DIAGRAM

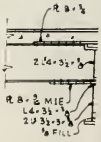


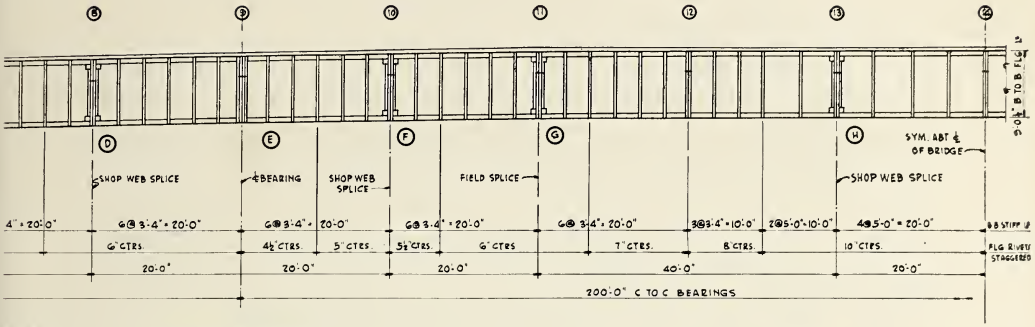
SHEAR DIAGRAM



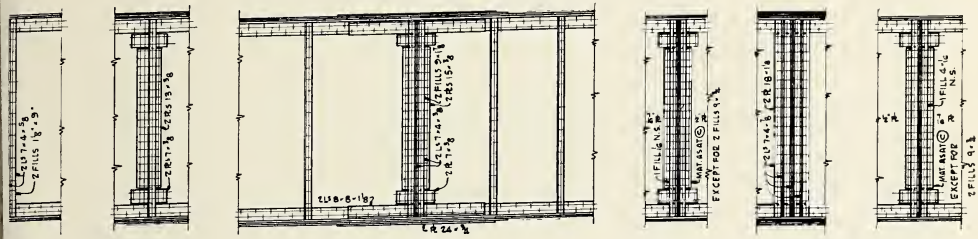
END SPAN

LONGITUDINAL SECTION AT STRINGERS

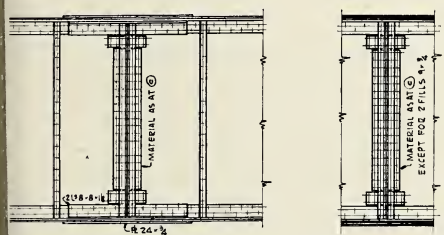
BEARING

INTERIOR



ECTION AT MAIN GIRDER



ZING AT A SPLICE AT B SPLICE AT C SPLICE AT D BEARING AT E SPLICE AT F

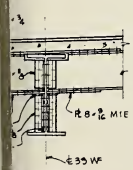


SPLICE AT G SPLICE AT H

GIRDER DETAILS

NOTES

INTERMEDIATE STIFFENERS 2 L 3½ × ½ CRIMPED ⅜" WITH ⅜" FILL PLATE EACH SIDE.
STIFFENERS AT FLOORBEAMS 4 L 7 × 4 × ⅝, EXCEPT AS NOTED, WITH 1⅛" FILLS EACH SIDE.



IOR SPANS

Figure 25

1. a. Structural steel	1,498,667 lbs. @	$ 0.18	$269,760	
b. Add for low alloy steel	173,740 lbs. @	0.02	3,475	
c. Total for structural steel				$273,235
2. Reinforcing steel	146,100 lbs. @	0.14		20,455
3. Concrete	615 C. Y. @	60.00		36,900
4. Asphalt surface	3,000 S. Y. @	1.00		3,000
				$333,590

"The corresponding estimate for the welded design included under Exhibit B is as follows:

1. a. Structural steel	1,119,237 lbs. @	$ 0.15	$167,885	
b. Add for low alloy steel	83,235 lbs. @	0.02	1,665	
c. Add for welds	5,641 lbs. @	12.00	67,690	
d. Total for structural steel				$237,240
2. Reinforcing steel	146,100 lbs. @	$ 0.14		20,455
3. Concrete	615 C. Y. @	60.00		36,900
4. Asphalt surface	3,000 S. Y. @	1.00		3,000
5. Total				$297,595

"These estimates indicate a saving in cost of $36,000. This is equal to 13 per cent of the structural steel work or 8 per cent of the total cost of the bridge superstructure.

"Comparison of quantities of steel between the two exhibits show a considerable saving in the amount of steel required for the welded design. Based on the total quantities of structural steel the welded design required 25 per cent less steel or comparing the riveted design with the welded design the riveted design required 33 per cent more steel. The major part of the saving is due to the reduction in connection materials. The riveted design required 17 per cent for such material whereas the welded design required only 9 per cent. The welded design required only 43 per cent of that required by the riveted design.

"In the case of the main material this proportion is considerably less and is due primarily to the reduction of the gross area by rivets. In the case of main material the welded structure required only 81.5 per cent of that required by the riveted structure. Furthermore, special low alloy steel for the welded structure was simpler to splice than for the riveted structure. Consequently, the welded structure

required only 48 per cent of that required by the riveted structure.

"Weight of welds for Exhibit B amounted to ½ per cent of the total weight of structural steel."

A. R. Cripe, Cleveland, Ohio, designed a four lane bridge of five spans having a total length of 552 ft. Both end spans are 96 ft. long and all of the three center spans are 120 ft. long. The girders are continuous over the piers, however, the second and fourth spans each have a suspended span of 62 ft.-6 in. and two cantilever spans of 28 ft.-9 in. The girders are 6 ft.-10½ in. deep except near the piers where the depth increases to 9 ft.-8½ in. at the piers. These dimensions and the span lengths are given in Figure 27.

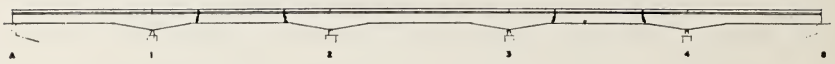
The bridge is divided and on either side two lines of girders 18 ft. apart support two of the four lanes plus a 5 ft. sidewalk. Figure 28 shows the manner in which the "rigid frame" floorbeams provide lateral support for the bottom flanges of the girders. The floorbeam spacing is 6 ft. and stringers are not required. The main transverse members of the "rigid frames" are 21 WF 62 sections between piers, 27 WF 102 sections at the piers, and 21 WF 82 sections at the abutments. The cantilever brackets, attached to the floorbeams to support the sidewalk, are spaced at 12 ft. centers.

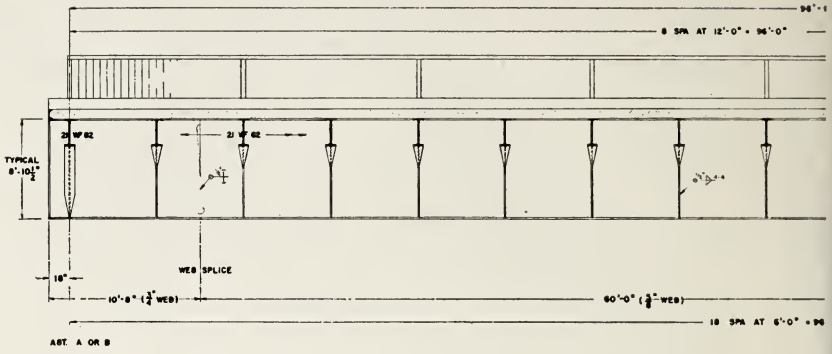
The flange plates for the girders vary in thickness and width from 18 in. by 1 in. plates, to 20 in. by 1¼ in. plates. The webs are ¾ in. thick near the supports and hinges and ⅝ in. thick elsewhere.

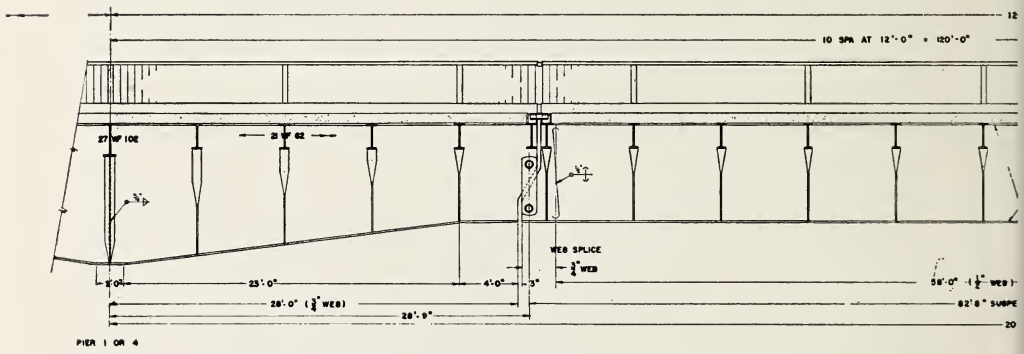
The structural steel for the riveted structure weighs 1,171,249 lb. for the girders, 541,076 lb. for the floor, 84,407 lb. for the bracing, and 13,955 lb. for miscellaneous material for a total of 1,810,687 lb. of steel requiring 35,000 rivets. For the welded bridge the structural steel needed is as follows: 927,856 lb. for the girder, 457,908 lb. for the floor, 60,340 lb. for stiffeners and miscellaneous parts to give a total of 1,446,104 lb. The shop welds weigh 8,171 lb. and the field welds 2,837 lb.

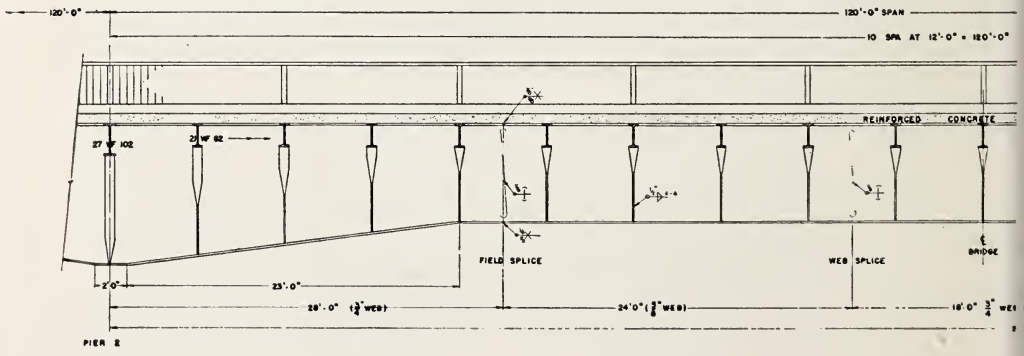
R. Reikenis, Baltimore, Maryland, chose the three span continuous bridge skewed at both ends as shown in Figure 29. The three spans are 92 ft., 115 ft., and 92 ft. to provide a total length of 299 ft. The four lane bridge is divided into two separate halves with each half supported by two lines of girders spaced 20 ft.-6 in. apart. Floorbeams span the distance between girders but are supported also at their mid-span by a longitudinal beam which itself is supported by cross frames every 23 ft. The floorbeam spacing is 5 ft.-9 in.

As his Exhibit A, Mr. Reikenis utilized a bridge recently built that has girders of constant depth (66¼ in. webs), This bridge


A
1
2
3
4
B


8 SPA AT 12'-0" = 96'-0"
TYPICAL
8'-10½"
WEB SPLICE
10'-8" (½" WEB)
60'-0" (⅜" WEB)
18 SPA AT 6'-0" = 96
ABT. A OR B


10 SPA AT 12'-0" = 120'-0"
WEB SPLICE
23'-0"
4'-0"
28'-0" (¾" WEB)
28'-9"
PIER 1 OR 4


120'-0" SPAN
10 SPA AT 12'-0" = 120'-0"
REINFORCED CONCRETE
FIELD SPLICE
WEB SPLICE
23'-0"
28'-0" (¾" WEB)
24'0" (⅜" WEB)
PIER 2

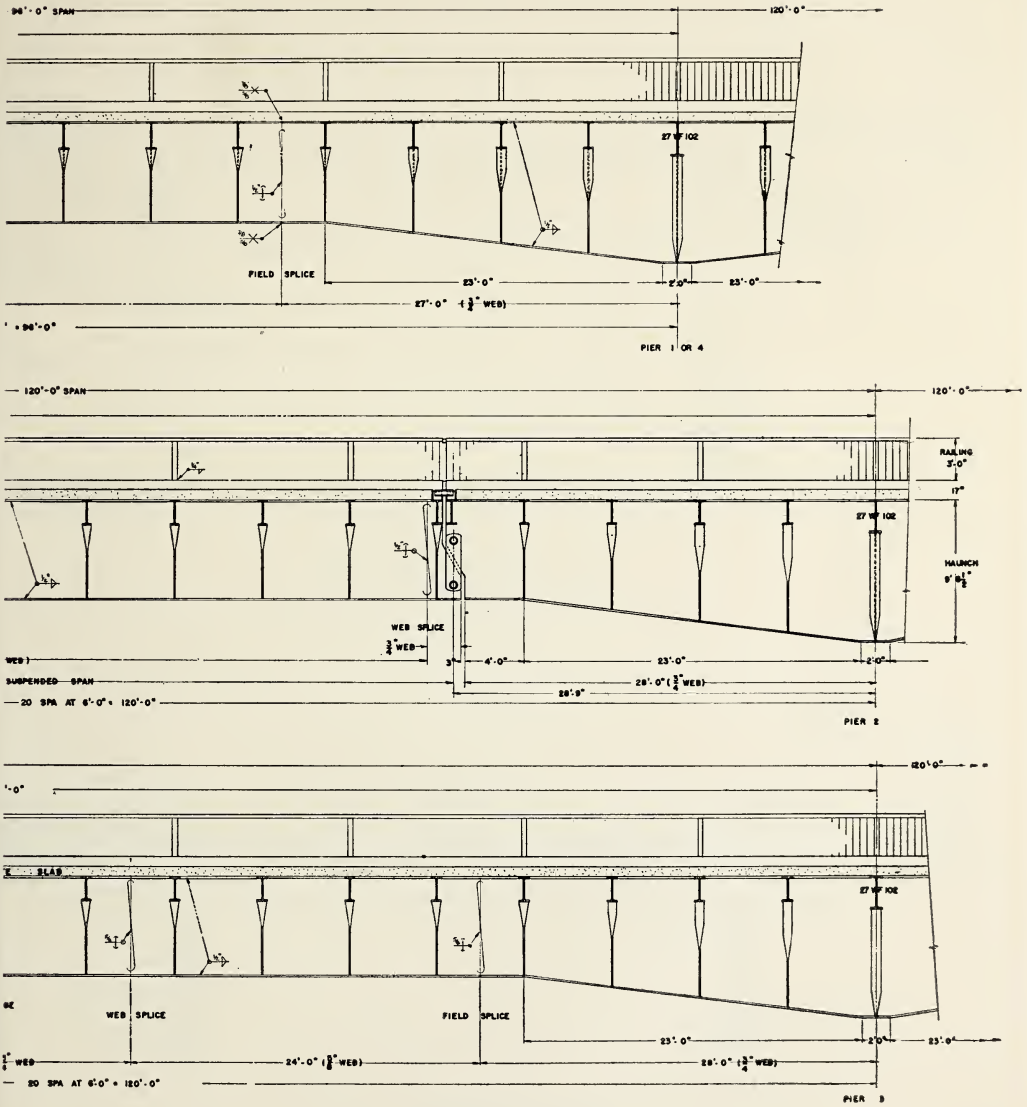
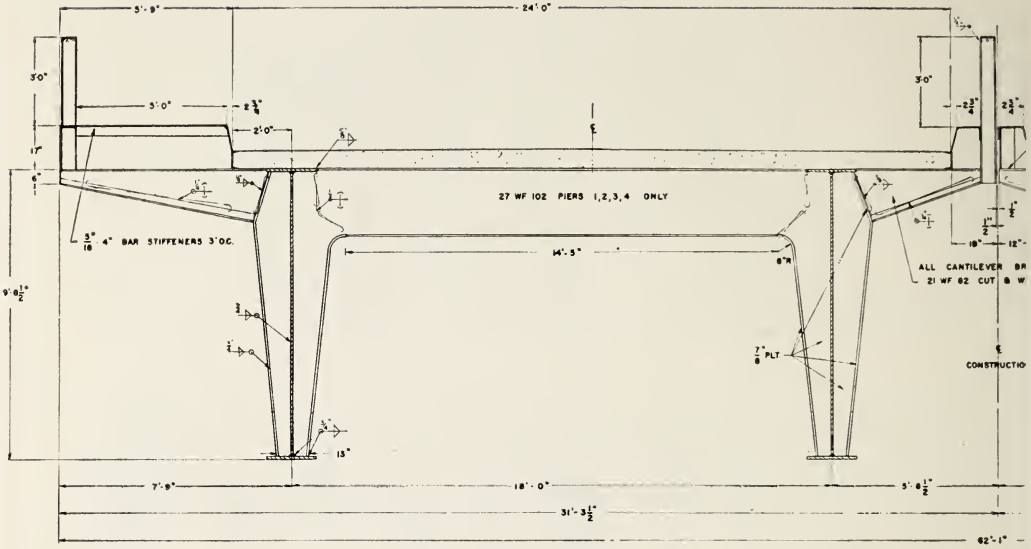
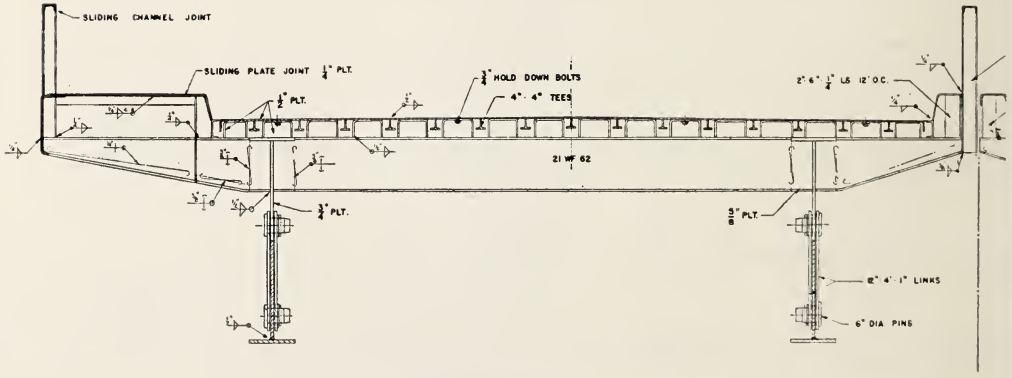

96'-0" SPAN
120'-0"
27 WF 102
FIELD SPLICE
23'-0"
2'-0"
23'-0"
27'-0" (¾" WEB)
PIER 1 OR 4
120'-0" SPAN
RAILING 3'-0"
17"
HAUNCH
WEB SPLICE
¾" WEB
4'-0"
23'-0"
28'-0" (¾" WEB)
SUSPENDED SPAN
20 SPA AT 6'-0" = 120'-0"
PIER 2
SLAB
WEB SPLICE
FIELD SPLICE
24'-0" (¾" WEB)
PIER 3

Figure 27

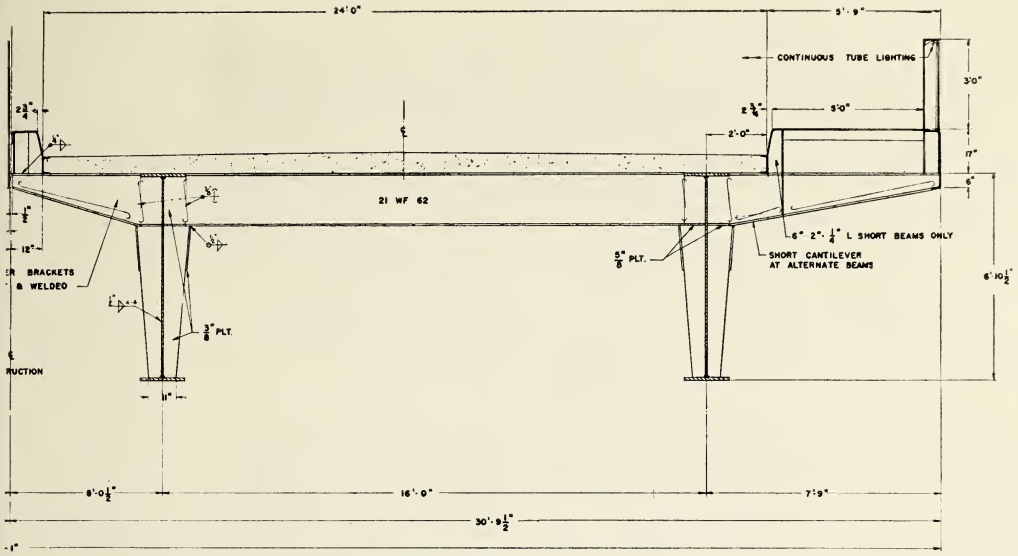


HALF SECTION AT PIER

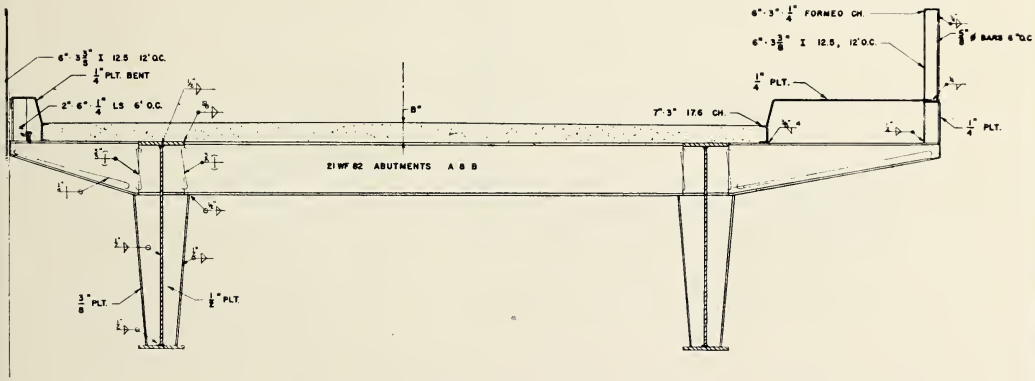


SECTION AT SUSPENDER

VIEW TOWARD CANTILEVER ARM

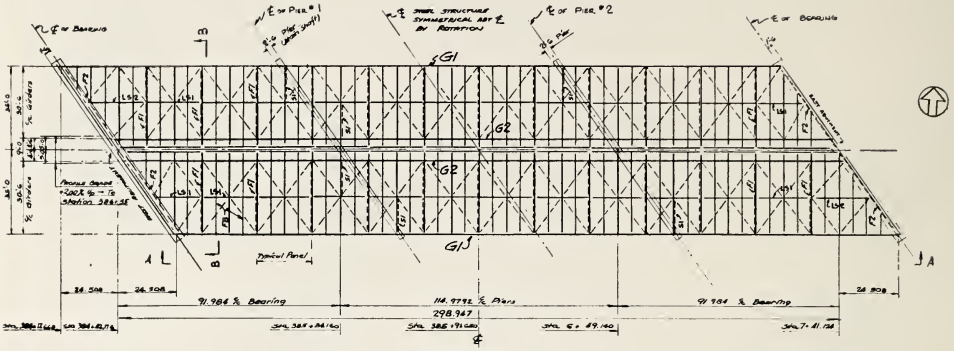


TYPICAL HALF SECTION



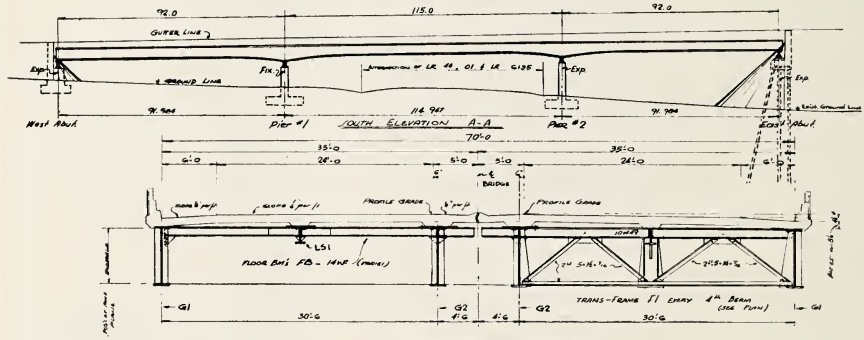
SECTION AT ABUTMENTS

Figure 28



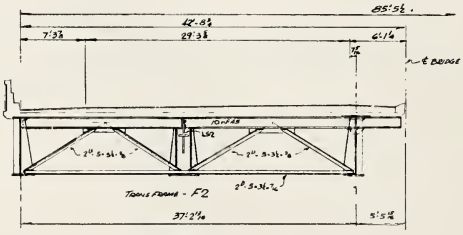
STEEL PLAN

ALL DIMENSIONS GIVEN IN HORIZONTAL PLANE



SOUTH ELEVATION A-A

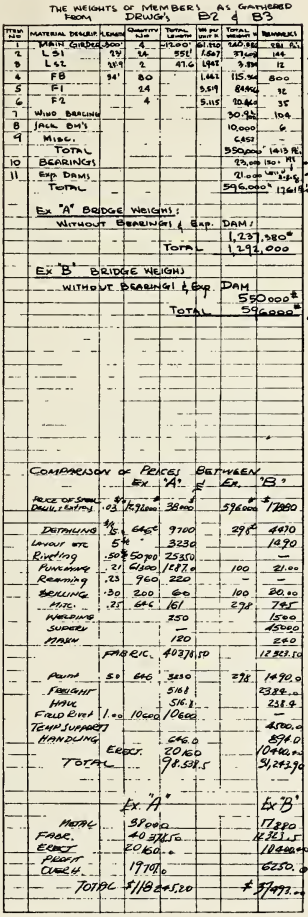
TYPICAL CROSS SECTION B-B



SECTION C-C

GENERAL NOTES

1. SPECIFICATIONS: A.A.S.H.T.O. Specifications, Std Specs for Highway Bridges & Std Specs for Welded Highway & Railway Bridges by American Welding Society
2. LIVE LOAD: E Loading H 20-S16-44 or Class M 18 Loading (Tandem or Each Pair of Trucks) when Specified Loading is not Outside Girders Carry Reaction of 16 Lanes, Inside Girders Carry a Minimum of 12 Lanes. Sidewalk Live Load 85 #/sq ft
3. ASSUMED DEAD LOAD: For Slab 8½" Reinforced Concrete 119 #/sq ft Spacing in Main Girders Calculated by Using Panel Loads of 5 ft Interval (Weights of Trans. Deck Beams) Outside Girders G1 Inside Girders G2
   Concrete 17/12 = 1.42 k/l.f. 17/12 = 1.42 k/l.f.
   Steel Trans. Deck Beams .178 k/l.f. .178 k/l.f.
   Misc. Est. .178 k/l.f. .178 k/l.f.
   Total 1.776 k/l.f. 1.776 k/l.f.
   Railing 0.200 k/l.f. 0.200 k/l.f.
   Wearing Surface 0.240 k/l.f. 0.240 k/l.f.
   Total 2.216 k/l.f. 2.216 k/l.f.
   Loading - G1 & G2 Design Information: Outside Girders Dist. Reaction Assumed as Load Per Lin. Ft. of Bridge Width with Load Density #2 # per c.f. Pavement to be Considered Part of ½ Cross Section Dead Load & ¾ Pavement Thickness
4. MATERIAL: Structural Steel ASTM A7 - 42 for Entire Symmetrical Built Structural Steel Is Hybrid Consisting of Main Elements of High Strength Steel ½" Plate
5. WORKMANSHIP: All Fab. and Erection Steel for Welded Units Shall Conform Throughout to Section C
6. ASSEMBLY: All Members to be Assembled and Prepared with Reasonable Care for Accurate Fitting of Parts. Shims are Placed between Splice Plate & Member Requiring Shims. Adjustment of Camber and Profile is Accomplished by Means of Shims Placed between Splice Plate & Member Requiring Adjustment. Minimum Thickness of Shims is ⅛". Shims Shall be Placed Between Splice Plate & Member Requiring Adjustment. Minimum Thickness of Shims is ⅛". Shims Shall be Placed Between Splice Plate & Member Requiring Adjustment. Minimum Thickness of Shims is ⅛".
7. PAINT: Shop: One Coat Red Lead in Oil (Min.) Dry & Inspected for Defects. Field: Two Coats, First Coat Red Lead in Linseed Oil (Min.) Dry & Inspected for Defects. Second Coat Red Lead in Linseed Oil (Min.) Dry & Inspected for Defects.
8. CAMBER: Main Girders Cambered for Dead Load Only



THE WEIGHTS OF MEMBERS AS GATHERED FROM DRWGS B2 & B3

ITEM NO	MATERIAL DESCRIP	LENGTH	QUANTITY PER	TOTAL LENGTH	WT PER UNIT ft	TOTAL WEIGHT #	REMARKS
1	MAIN GIRDER	300'	4	1200'	201.150	240,984	201 A.L.
2	L 5½	49'	56	2744	4.687	87,400	146
3	L 6½	38'9	2	77.6	6.947	9,394	12
4	FB	94'	80		1.461	115,360	800
5	F1		24		3.519	84,864	72
6	F2		4		5.115	10,464	35
7	WIND BRACING					30,912	104
8	JACK DM'S					10,000	6
9	MISC.					CASE	—
	TOTAL					550,000#	1413 A.L.
10	BEARINGS					23,000 (80) ea	
11	EXP DAMS					21,000 (40) ea	
	TOTAL					596,000#	1761 A.L.

Ex "A" BRIDGE WEIGHTS:

WITHOUT BEARINGS & EXP. DAMS	1,237,380#
TOTAL	1,292,000

Ex "B" BRIDGE WEIGHTS

WITHOUT BEARINGS & EXP. DAMS	550,000#
TOTAL	596,000#

COMPARISON OF PRICES BETWEEN Ex. "A" & Ex. "B"

		Ex "A"		Ex. "B"	
PRICE OF STEEL DELIV. & ERECT.	.03	#/M 38,000	$ 1140	#/M 596,000	$ 17,880
DETAILING	5/16	646	9700	298	4470
LAYOUT ETC	5%		3236		1490
RIVETING	.50	5070	2535		—
PUNCHING	.21	6500	1287	100	21.00
REAMING	.23	960	220	—	—
BOLTING	.30	200	60	100	30.00
MISC.	.25	646	161	298	745
WELDING			750		1500
SURVEY			—		4500
PLAN			120		240
		FAB ETC.	40378.50		12363.00
PAINT	.50	646	3230	298	1490.00
FREIGHT			5168		2388.00
HAUL			516.8		238.9
FIELD RIVET	1.00	10,000	10,000		4500.00
TEMP SUPPORT					894.0
HANDLING			646.0		10440.00
ERECT.			20160		
TOTAL			98,508.5		31,242.90
		Ex "A"			Ex "B"
METAL		38,000			17,880
FABR.		40 3/4%			12,363.00
ERECT		20160			10440.00
PAINT & CLEAN		1970			6250.00
TOTAL		$118,245.20			$ 47,933.00

TOTAL WEIGHTS

Ex "A" — 1292000 # SEE STEEL

Same with Comp Action Herein Called "A" = 1108000# " "

Ex "B" 596000# " "

Same without Cond Action Herein Called "B" 770000# " "

$\frac{A'}{A} = \frac{1108000}{1292000} = .855$ or 14.5% saving against Ex. "A"

Same as Ex "A" $\frac{B}{A} = \frac{596000}{1292000} = .46$ or 54% saving as against Ex "A"

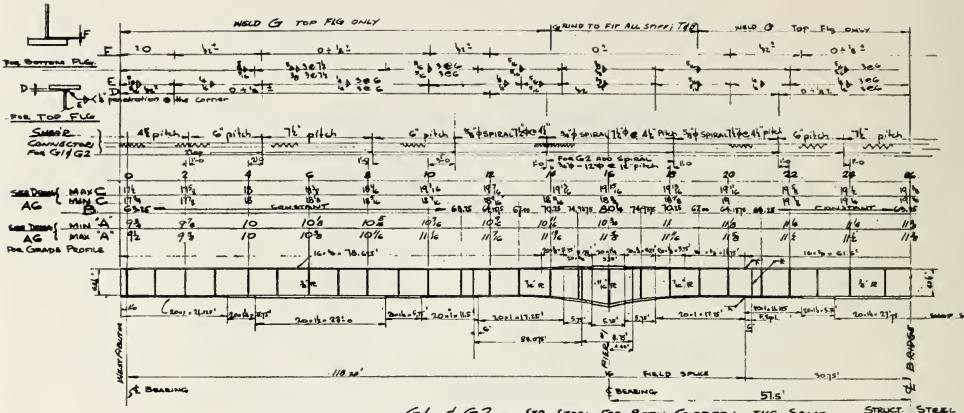
Same as Ex "A" $\frac{B'}{A} = \frac{770000}{1292000} = .595$ or 40.5% saving against "A"

$\frac{B}{B'} = \frac{596000}{770000} = .775$ or 22.5% saving against "B"

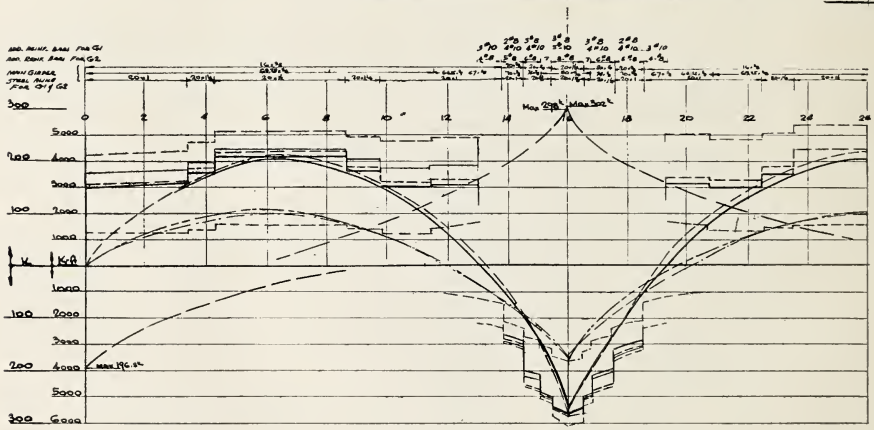
$\frac{B}{A'} = \frac{596000}{1108000} = .505$ or 49.5% sav. against A'

STR. STEEL CONTRACT TO DL.: $\frac{550000}{1237380} = .445$ or 55.5% LESS $\frac{B}{A}$!

Figure 29



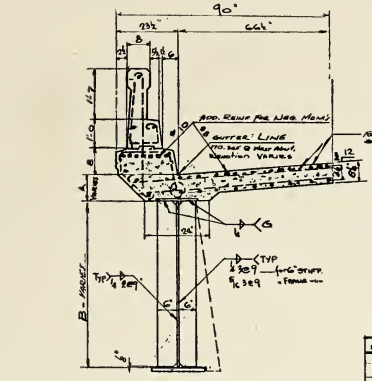
G1 & G2 - STR. STEEL FOR BOTH GIRDERS THE SAME



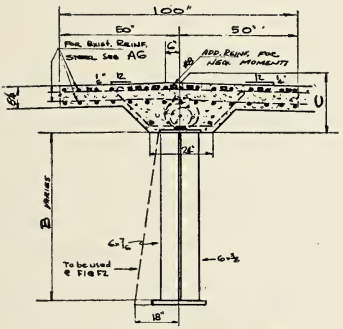
NOTE: FOR ARRANGE OF RAILING MOMENTS SEE FOR G1 - SHEET D1. NOTE DIFF OF LOAD THROUGH, & DEAD LOAD AND AMOUNT RESISTING RAILING ADD G2 - SHEET D2 WITH HYPOTHESES MARKED & MORE CURVE

FOR MOMENT CURVES:
- ——— ALL MOMENT FOR G1
- — — — DL — — G1
- — — — ALL MOMENT FOR G2
- — · — · — DL — — G2

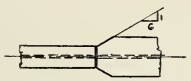
FOR RESISTING MOMENT:
- ——— FOR TENSION FLG AT MOMENT G1
- ——— FOR COMPRESSION CONCR. — — G1 (REINF. STEEL & LIVE MOM.)
- — — — FOR TENSION FLG AT MOMENT G2
- — — — FOR COMPRESSION CONCR — — G2 (REINFORCEMENT & LIVE MOM.)
- — — — STR. STEEL ONLY (ERECTION)



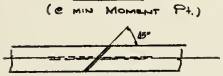
TYPICAL SECTION FOR G1



TYPICAL SECTION FOR G2



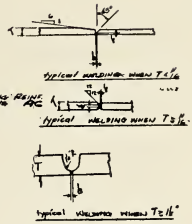
TYPICAL CHANGE IN WIDTH OF FLG (@ MIN MOMENT Pt.)



TYPICAL BUTT JOINT CUT OF FLG

FOR CONNECTIONS TO THE GIRDERS SEE B3

DRWG. No B2



No	MATERIAL	LENGTH	No Pcs	THICKNESS	WT/FT	Total wt	
1	WEB R 63"x 3/8	66'-11"	2	199.25'	80.5	16.04	
2	do	66.5	1	66.5'	do	4.33	
3	WEB R 76" x 7/16	82'-11"	2	44.14'	76.6	3.39	
4	WEB R 80" x 1/2	8'-11	2	17.5	106.25	1.39	
5	Flg. R 16" x 3/8	76.25	2	153.16'	20.4	3.20	
6	do	61.5	1	61.5'	do	1.25	
7	do	11.75	2	23.5'	do	.48	
8	Flg. R 20" x 3/8	5.75	4	23.0	25.5	.58	
9	Flg. R 20" x 5/8	5.75	4	23.0	51.0	1.17	
10	Flg. R 20" x 1 1/4	5.75	2	11.5'	85.0	.97	
11	Flg. R 20" x 1	21.125	2	42.25	68.0	2.87	
12	do	11.5	2	23.0	do	1.56	
13	do	17.25	4	69.0	do	4.55	
14	do	11.25	2	22.25	40	1.51	
15	Flg. R 20" x 1/4	5.75	10	57.5'	8.50	4.88	
16	Flg. R 20" x 1 1/4	23.0	2	46.0	102.0	4.69	
17	do	21.25	1	21.25	do	2.83	
18	Flg. R 20" x 5/8	5.75	2	11.5'	110	1.26	
	MAIN MATL / GIRDER		47			82.47	
19	Stiffs 6" x 3/8	5'-3	41	215.0	7.65	1.64	
20	do 6" x 7/16	6'-0	6	24.1'	8.93	.21	
21	do 6" x 1/2	5'-3	3	15.75	10.2	.16	
22	do 6" x 5/8	5'-3	2	10.5'	12.3	.16	
23	do 6" x 1	6'-0	1	6.0'	20.4	.12	
24	do 12" x 1	6'-0	1	6.0'	40.8	.24	cut from 24" wide
25	do 12" x 3/4	5'-3	2	10.5'	30.6	.32	24" wide
26	do 12" x 1/2	5'-3	16	84.0	20.4	1.71	Est. two end stiffs @ same
27	do 12" x 7/16	16.3	30	197.5	17.86	3.51	
28	do 12" x 3/8	6'-0	2	12.0	15.34	.41	
29	LR 6" x 3/8	0'-6	24	6.0	6.38	.06	
30	LR 3" x 3/16	0'-3	24	3.00	3.24	.01	
31	R 5" x 3/8	0'-6	36	18.0	12.8	.23	
32	L-6" x 6" x 3/8	0'-6	36	18.0	28.7	.51	
33	R 9" x 3/8	2'-6	12	30'	11.5	.34	
	TOTAL		281			61.18	

	WELD TYPE	LENGTH	No Pcs	TOTAL LG	Cost/ft	Total Cost
34	1/4	671'	.16	8138	.055	28.16
35	5/16	962'	.20	1246	.059	21.36
36	3/8	3008'	.30	9240	.064	3.35
37	7/16	16'	.48	864	1107	11.97
38	1/2	12'	.48	638	.295	8.54
39	5/8	11'	.61	6.91	.18	1.24
40	3/4	14'	.71	9.94	.33	4.66
41	1"	6'	1.47	8.81	.713	1.63
42	1 1/4	20'	1.98	40.76	.411	8.42
43	1 1/2	7'	2.78	19.46	.887	3.23
44	2"	7'	4.9	32.90	.841	5.54
45	2 1/2	12'	7.55	6.6	.990	1.17
	TOTAL			364.45		$118.76

Figure 30

has 1,292,000 lb. of structural steel. His welded bridge contains 770,000 lb. of structural steel. As an additional comparison, he designed both the riveted and welded structures to act compositely with the reinforced concrete slab. The resulting weights for structural steel are 1,108,000 lb. and 596,000 lb., respectively. Both welded girders varied in depth as shown in Figure 30. The girders webs are 63¼ in. deep at the abutments and at the middle of the center span and are 80 in. deep at both piers. The webs vary in thickness from ⅜ in. to 7/16 in. to 11/16 in.

Figure 29 contains a cost and weight summary. Figure 30 shows some of the welded details and a material list and a list of welds. The total weight of welds is 385 lb.

Continuous Girder Bridges—Prestressed

T. C. Kavanagh and Leo Coff, both of New York, New York, presented the design of a three span continuous bridge with a total length of 990 ft. The end spans are 270 ft. each and the center span is 450 ft. long. The main longitudinal members are three deck girders that become two box girders for 180 ft. of length (90 ft. each side) adjacent to the piers as shown in Figure 31. The top of the boxes is the 9 in. concrete slab which acts integrally with the structural steel. The bottom of the boxes consists of a ¾ in. plate stiffened with 22 longitudinal beams (10 WF 29). At the piers, the box girders are 17 ft.-9 in. deep. Midway between piers and at the abutments, the plate girders are 9 ft.-9 in. deep.

The roadway is 36 ft. wide between curbs and there are 3 ft. sidewalks on both sides. Cross frames occur every 30 ft.; however, there are no stringers or floorbeams. The composite slab spans the 16 ft. between girders (or webs of the boxes). The slab is haunched at these three lines of support.

Not only are the slab and girders of composite construction, but both the reinforced concrete slab and the steel girders are prestressed by longitudinal cables. The vertical location of the cables is shown in Figure 31. The lateral positions of the prestressing cables are indicated in the cross sections of Figure 32.

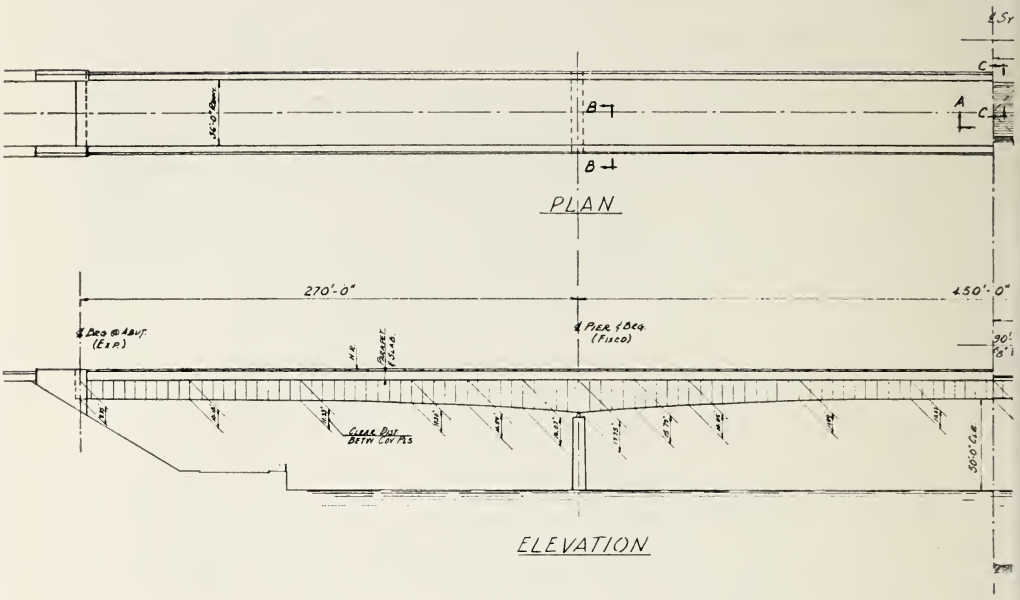
For the riveted and welded girders and boxes, the web thickness changes from ¾ in. at the piers to 11/16 in. to ⅝ in. at the abutments and mid-way between piers. The welded girder has a single 8 x 8 x 1 in. angle and a plate as a flange. This one angle

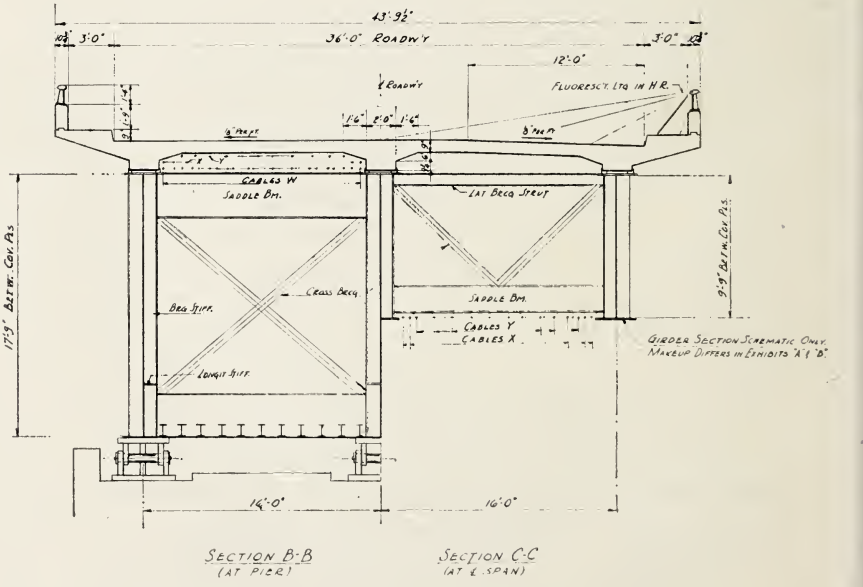
is rotated from the normal position to become a vee as shown in Figure 32. Figure 33 shows some of the details for the riveted structure.

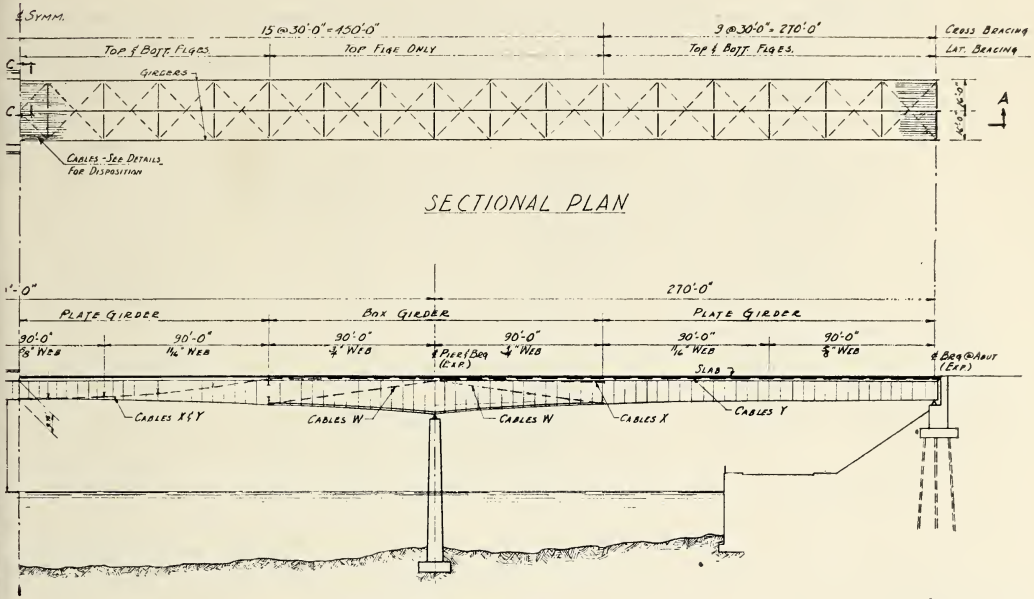
Messrs. Kavanagh and Coff discussed their design in the following manner:

"Where steel conservation is a major factor in highway bridge design it appears proper to concentrate attention on bridges of the girder and beam type, which constitute by far the largest tonnage of all classes of bridge construction. As an example, the recent New Jersey Turnpike design [see 'New Jersey Tuhnpike Issue', *Civil Engineering,* Vol. 22, No. 1, Jan. 1952, and F. S. Merritt, 'Longest Plate Girders are Twins', *Engineering News-Record,* Jan. 17, 1952, pp. 30-33], with a total of close to 200,000 tons of steel, specified the avoidance of truss construction of any type in favor of rolled beams or built-up girders—even to the extent of incorporating its two longest spans of record plate girder length (375 ft.). The importance of the girder type of construction extends also to most other classes of bridge construction where it is employed in floor systems; to suspension bridges and arch bridges, where it is used in the form of stiffening girders; to steel rigid frame bridges, which are essentially modified plate girders; and to movable bridges of all kinds.

"Although girder bridges have formed a part of our structural knowledge for over a century now, or since the time of the eminent George Stephenson, their use appears to have been restricted for a good part of that time to the conventional, stolid and somewhat uninspiring railroad bridges. It is really only in the last twenty years or so that a new period of bridge construction emerged, characterized by steel beam and girder highway bridges and overpasses which have rapidly displaced most other types of construction in this field. Perhaps, without realization of the transition, we find ourselves in the midst of what might be termed the *plate girder era,* with newer and larger structures coming up each year. Thus, in this country we have seen following each other in rapid succession the Thos. A. Edison Bridge (1940) of 250 ft. span, the Lakefront Bridge in Cleveland (1940) of 271 ft. span, the Charter Oak Bridge of 300 ft. span (1942). the Harlem River pedestrian bridge of 330 ft. span (1950), and the Passaic and Hackensack River Bridges of 375 ft. span (1951), with spans of 400 to 450 ft. in contemplation on the New Jersey Turnpike extension and on the Tappan Zee Bridge approaches. In the same period there have appeared on the continent of Europe such monumental girder spans


PLAN
270'-0"
450'-0"
ELEVATION


43'-9½"
3'-0"
36'-0" ROADWAY
3'-0"
12'-0"
FLUORESCT. LTG IN H.R.
CABLES W
SADDLE BM.
LAT BRCG STRUT
CROSS BRCG
BRG STIFF.
LONGIT STIFF.
SADDLE BM.
CABLES Y
CABLES X
GIRDER SECTION SCHEMATIC ONLY. MAKEUP DIFFERS IN EXHIBITS 'A' & 'B'.
14'-0"
16'-0"
SECTION B-B (AT PIER)
SECTION C-C (AT ℄ SPAN)
HALF-SECTIONS



General Notes:

Layout on this sheet applies to both Exhibit "A" (Riveted) and Exhibit "B" (Welded) Designs. For Details see accompanying Dwgs.

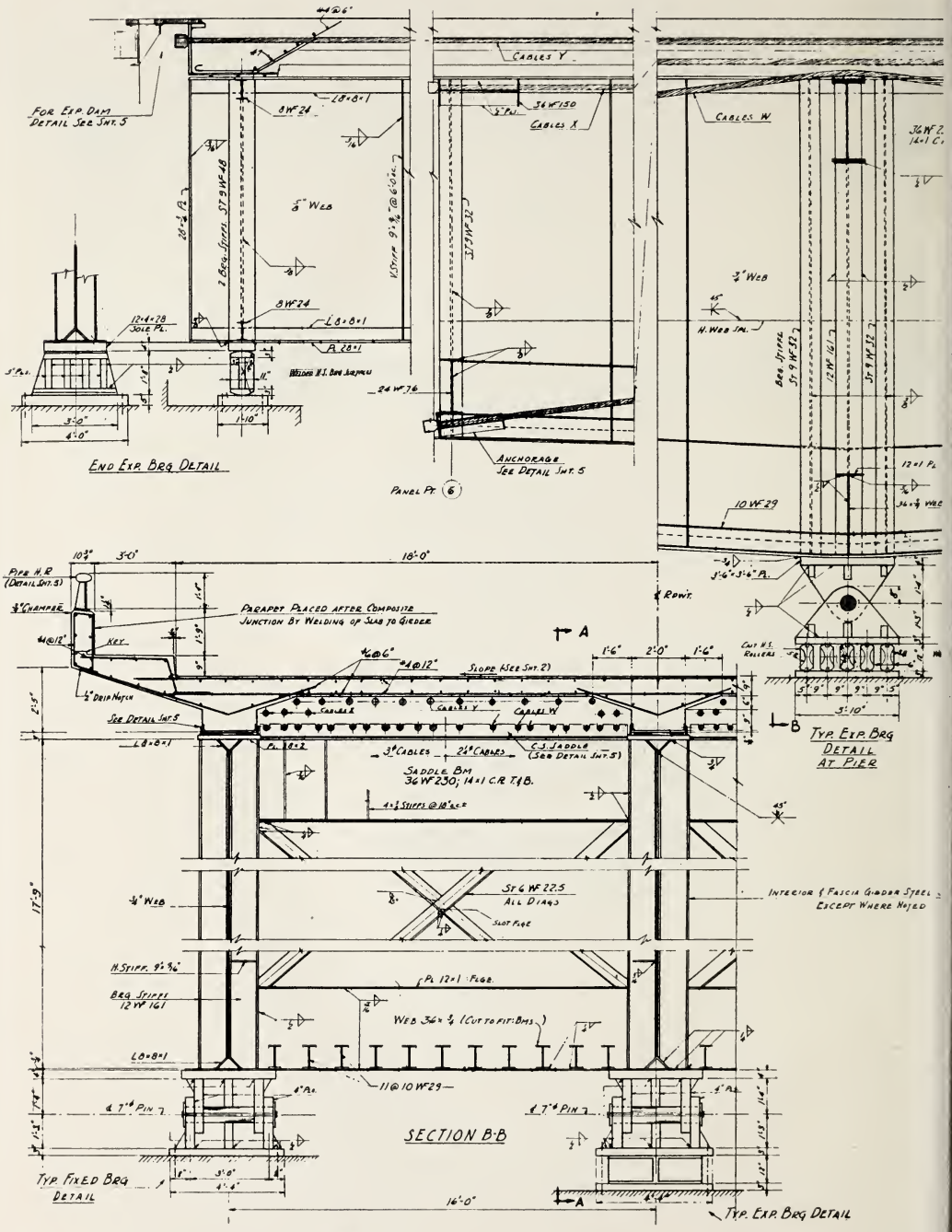
Materials:

Steel: Structural Steel ASTM-A7-46.
Cables: 2½"ϕ & 3"ϕ Bridge Cable (Galv.).
Concrete: 5000 P.S.I. in 28 Days.

Loading: H20-S16-44.

Specifications: AASHO 1949.
AWS (Bridges) 1947

Figure 31


FOR EXP. DAM DETAIL SEE SHT. 5
CABLES Y
CABLES X
CABLES W
36 WF 150
8 WF 24
L8×8×1
5" WEB
2 BRG. STIFFS. ST 9 WF 48
ST 9 WF 32
12"×28" SOLE PL.
3" PL.
1½"
4"
Milled N.T. Bed Surface
24 WF 76
3'-0"
4'-0"
1'-10"
END EXP. BRG. DETAIL
ANCHORAGE SEE DETAIL SHT. 5
PANEL PT. (6)
3" WEB
45°
H. WEB INCL.
BRG. STIFFS. ST 9 WF 32
12 WF 161
12"×1" PL.
10 WF 29
3'-6"×3'-6" PL.
℄ ROWT.
CUT W.S. ROLLERS
3'-10"
TYP. EXP. BRG. DETAIL AT PIER
10¾"
3'-0"
18'-0"
PIPE H.R (DETAIL SHT. 5)
¾" CHAMFER
PARAPET PLACED AFTER COMPOSITE JUNCTION BY WELDING OF SLAB TO GIRDER
KEY
4@12"
4@6"
SLOPE (SEE SHT. 2)
1'-6"
2'-0"
1'-6"
½" DRIP NOTCH
SEE DETAIL SHT. 5
CABLES Y
CABLES W
L8×8×1
PL. 12×2
3" CABLES
2½" CABLES
C.I. SADDLE (SEE DETAIL SHT. 5)
SADDLE BM 36 WF 230; 14×1 C.R. T & B.
4×⅝ STIFFS @ 10" CLR
¾" WEB
ST. 6 WF 22.5 ALL DIAGS
SLOT FLGE
INTERIOR & FASCIA GIRDER STEEL EXCEPT WHERE NOTED
H. STIFF. 9'-9⅜"
BRG. STIFFS 12 WF 161
PL. 17×1 FLGE.
WEB 36×⅜ (CUT TO FIT BMS)
11@10 WF 29
6" PL.
℄ 7"ø PIN
SECTION B-B
TYP. FIXED BRG. DETAIL
16'-0"
TYP. EXP. BRG. DETAIL

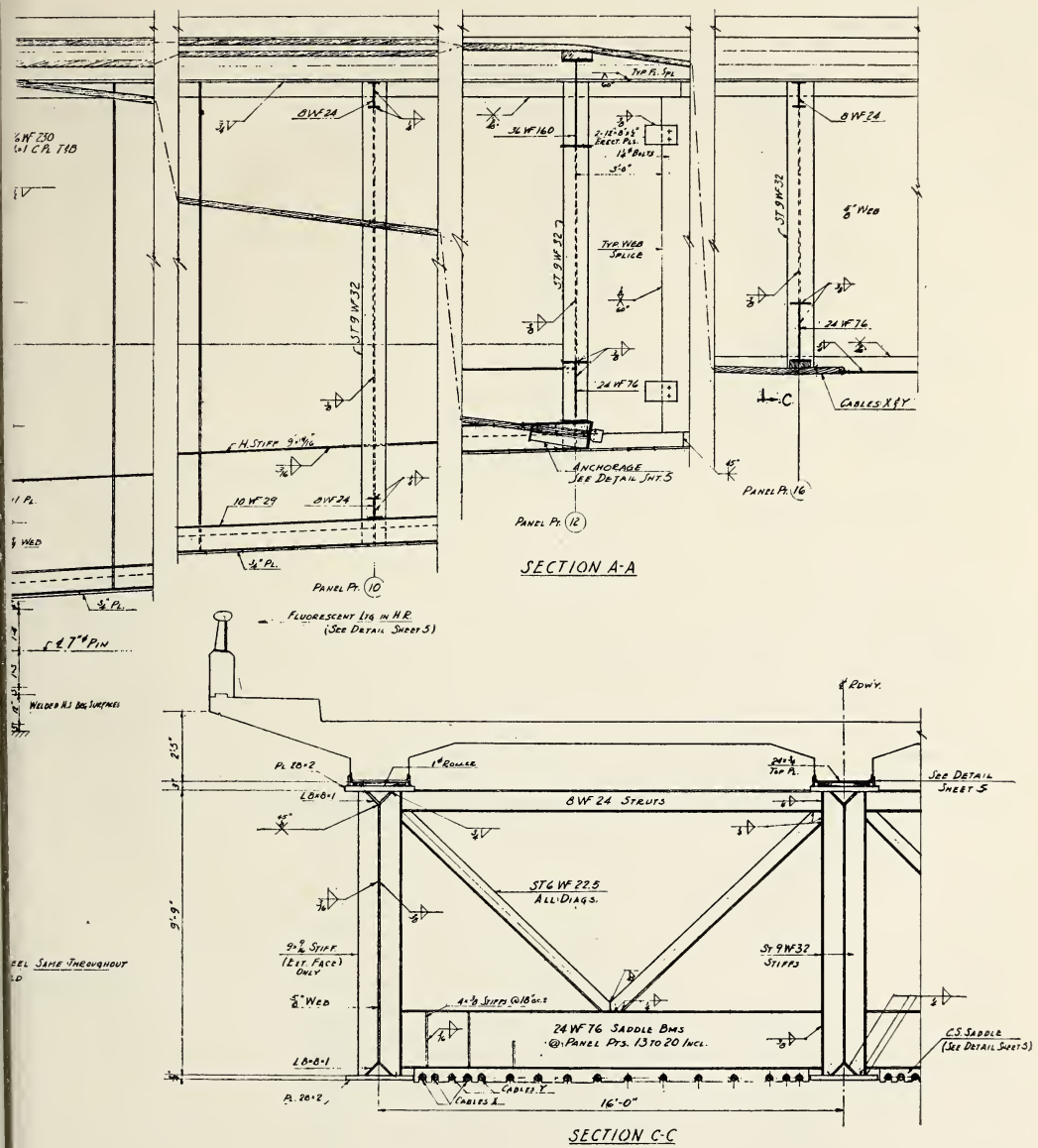
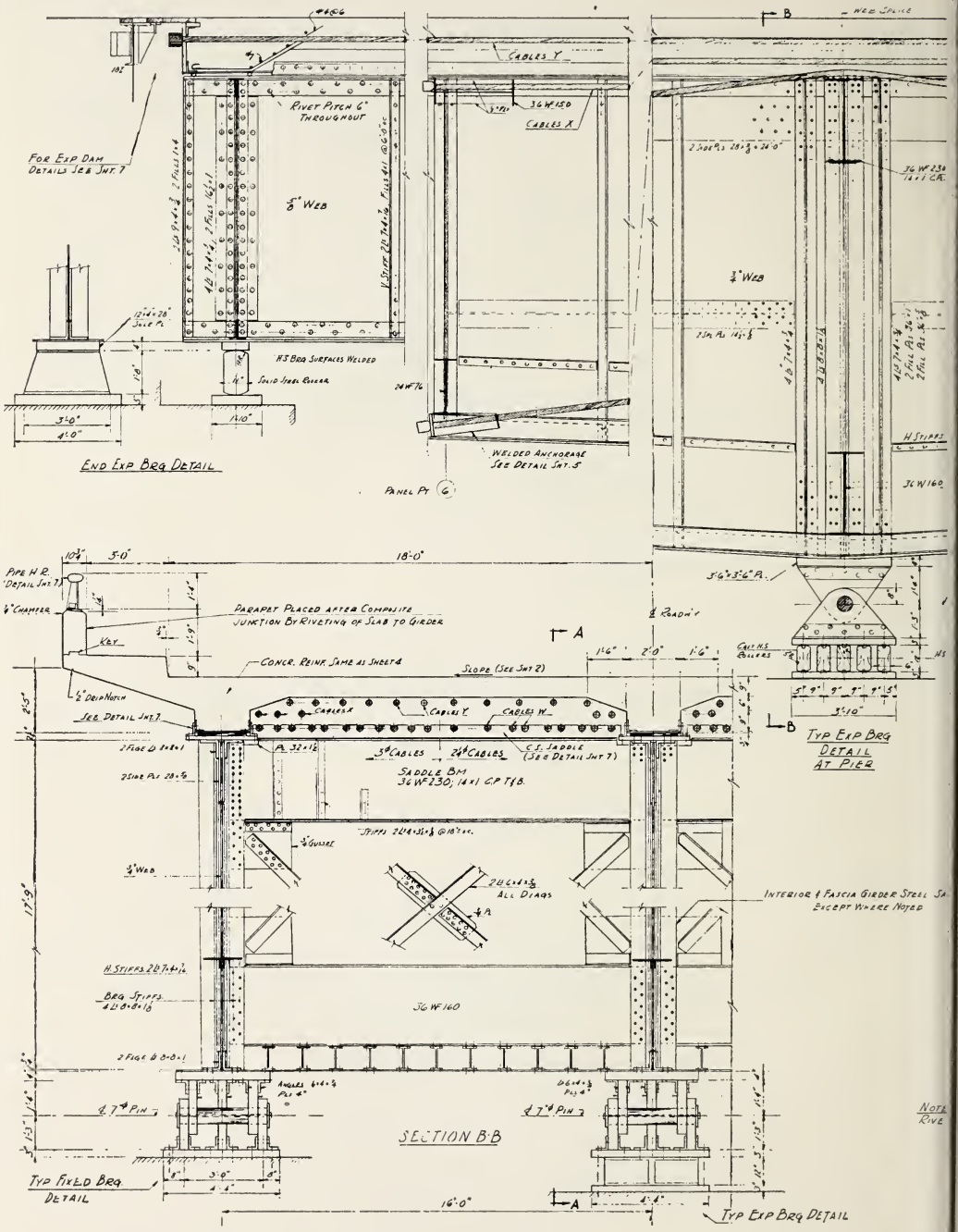

8 WF 24
ST 9 WF 32
TYP. WEB SPLICE
24 WF 76
CABLES X & Y
ANCHORAGE SEE DETAIL SHT. 5
PANEL PT. 12
PANEL PT. 16
PANEL PT. 10
10 WF 29
H STIFF 9"×⅝"
SECTION A-A
FLUORESCENT LIG. IN H.R. (See Detail Sheet 5)
1ST ROLLER
8 WF 24 STRUTS
STG WF 22.5 ALL DIAGS.
9"×⅝" STIFF (EXT. FACE) ONLY
ST 9 WF 32 STIFFS
24 WF 76 SADDLE BMS @ PANEL PTS. 13 TO 20 INCL.
C.S. SADDLE (See Detail Sheet 5)
SEE DETAIL SHEET 5
CABLES Y
CABLES X
16'-0"
SECTION C-C

Figure 32


RIVET PITCH 6" THROUGHOUT
FOR EXP DAM DETAILS SEE SHT 7
5/8" WEB
12"x26" SOLE PL
H.S. BRG SURFACES WELDED
4" SOLID STEEL RocKER
3'-0"
4'-0"
1'-10"
END EXP BRG DETAIL
CABLES Y
36 WF 150
CABLES X
WELDED ANCHORAGE SEE DETAIL SHT. 5
PANEL PT 6
WEB SPLICE
36 WF 230 16 x 1 CR.
7/8" WEB
H STIFFS
36 WF 160
5'6"x3'6" PL
CAST H.S. ROLLERS
5'-9" 9" 9" 9" 5"
3'-10"
TYP EXP BRG DETAIL AT PIER
18'-0"
PIPE H.R. DETAIL SHT 7
3/4" CHAMFER
PARAPET PLACED AFTER COMPLETE JUNCTION BY RIVETING OF SLAB TO GIRDER
KEY
CONCR. REINF. SAME AS SHEET 4
SLOPE (SEE SHT 2)
1/2" DRIP NOTCH
SEE DETAIL SHT 7
CABLES X
CABLES Y
CABLES W
3 1/2" CABLES
2 1/2" CABLES
C.I. SADDLE (SEE DETAIL SHT 7)
SADDLE BM 36 WF 230, 14 x 1 C.P. T & B.
STIFFS 2L 4 x 3 1/2 x 5/8 @ 18" c.c.
5/8" GUSSET
3/4" WEB
2 L 6 x 4 x 3/4 ALL DIAGS
INTERIOR & FASCIA GIRDER STEEL SA. EXCEPT WHERE NOTED
H. STIFFS 2 B 7 x 1/2
BRG STIFFS 4 L 8 x 8 x 1 1/8
36 WF 160
4 7/8" PIN
SECTION B-B
TYP FIXED BRG DETAIL
16'-0"
TYP EXP BRG DETAIL

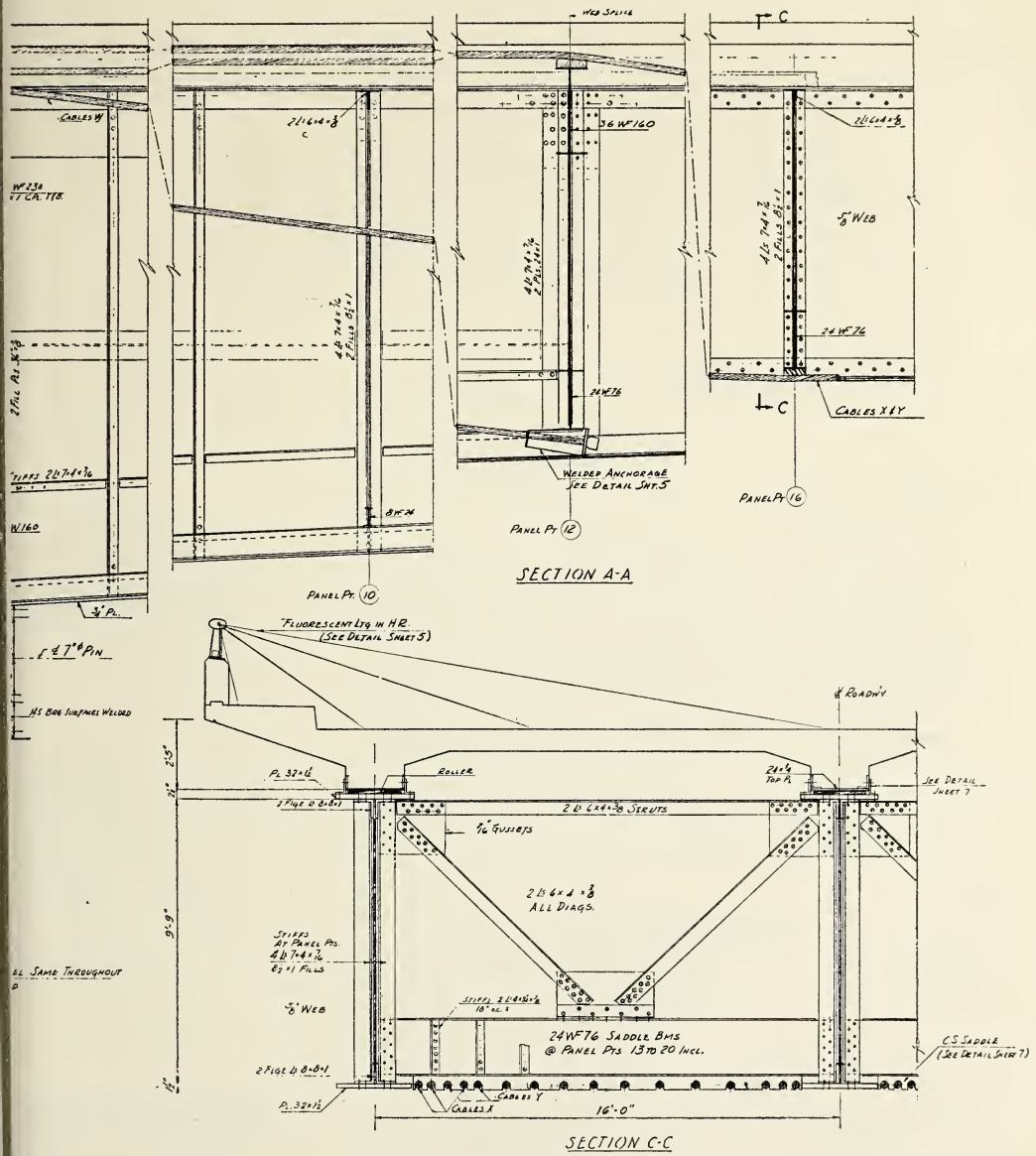
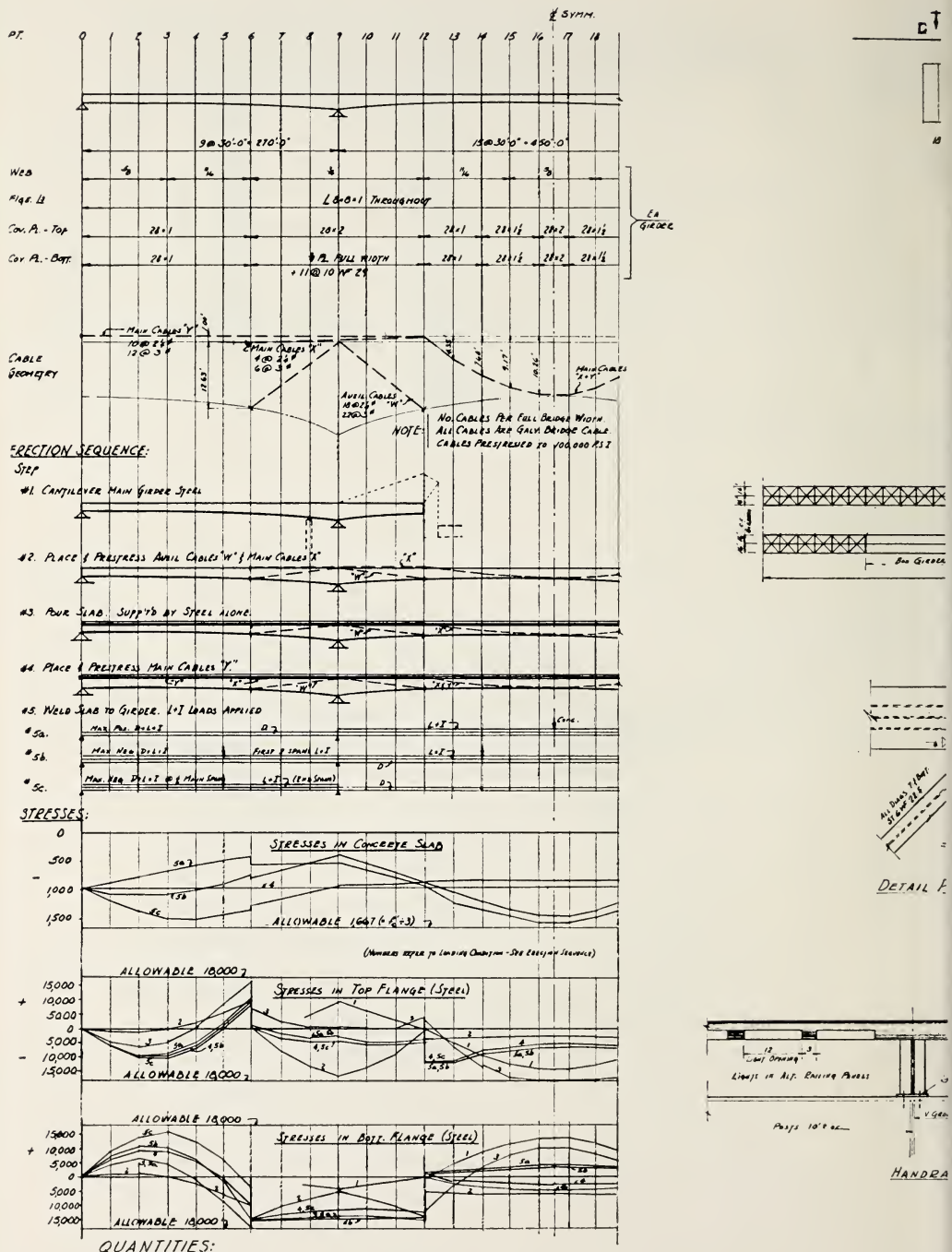

Web Splice
36 WF 160
24 WF 76
Welded Anchorage See Detail Sht. 5
Panel Pt. 12
Panel Pt. 16
Panel Pt. 10
Cables X & Y
5/8" Web
SECTION A-A
"Fluorescent Ltg in H.R. (See Detail Sheet 5)
℄ Roadway
Roller
2 L 6 x 4 x 3/8 Struts
5/16" Gussets
2 L 6 x 4 x 3/8 All Diags.
24WF76 Saddle Bms @ Panel Pts 13 to 20 Incl.
C.S. Saddle (See Detail Sheet 7)
Cables Y
Cables X
16'-0"
SECTION C-C
Note:
Rivets 1" except where noted

Figure 33

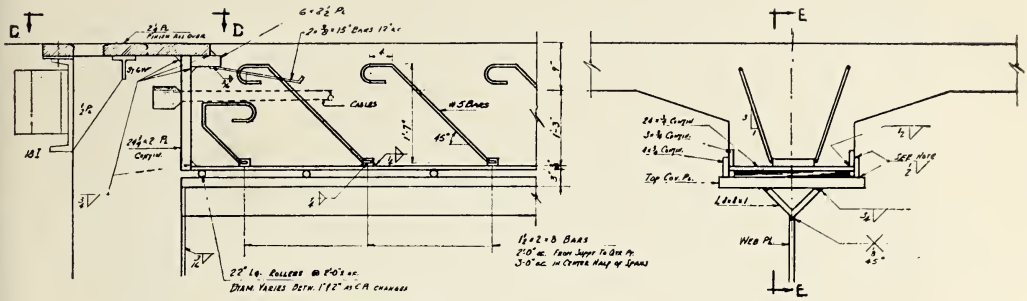

℄ SYMM.
P.T. 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
9 @ 30'-0" = 270'-0"
15 @ 30'-0" = 450'-0"
WEB
FLGE. LL
L & 0 + 1 THROUGHOUT
COV. PL. - TOP
COV. PL. - BOTT.
℄ PL. FULL WIDTH
+ 11 @ 10 W 29
EA GIRDER
CABLE GEOMETRY
MAIN CABLES "Y"
10 @ 2 ℄
12 @ 3 ℄
MAIN CABLES "X"
4 @ 2 ℄
6 @ 3 ℄
AUXIL. CABLES "W"
NOTE: NO CABLES PER FULL BRIDGE WIDTH
ALL CABLES ARE GALV BRIDGE CABLE
CABLES PRESTRESSED TO 100,000 PSI
ERECTION SEQUENCE:
STEP
#1. CANTILEVER MAIN GIRDER STEEL
#2. PLACE & PRESTRESS AUXIL CABLES "W" & MAIN CABLES "X"
#3. POUR SLAB SUPP'D BY STEEL ALONE
#4. PLACE & PRESTRESS MAIN CABLES "Y"
#5. WELD SLAB TO GIRDER. L+I LOADS APPLIED
#5a.
#5b.
#5c.
STRESSES:
STRESSES IN CONCRETE SLAB
ALLOWABLE 1647 (f'c+3)
(NUMBERS REFER TO LOADING CONDITION - SEE SEQUENCE)
ALLOWABLE 18,000
STRESSES IN TOP FLANGE (STEEL)
ALLOWABLE 18,000
STRESSES IN BOTT. FLANGE (STEEL)
ALLOWABLE 18,000

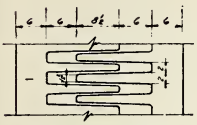
QUANTITIES:

CONCRETE	1840 CU YDS	STRUCTURAL STEEL A-7	3,547,400 LBS.
REINF STEEL (STR. GRADE)	383,700 LBS	GALV BRIDGE CABLE	382,000 LBS
CABLE FITTINGS	8,640 LBS.	CAST STEEL SADDLES	67,008 LBS.

DETAIL F

HANDRA





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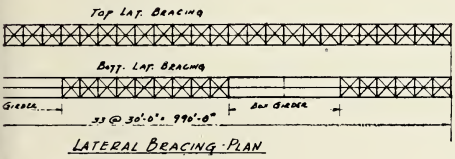
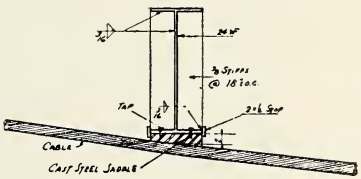
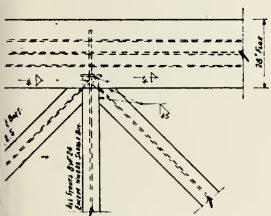
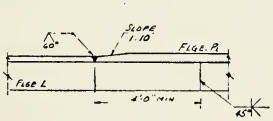
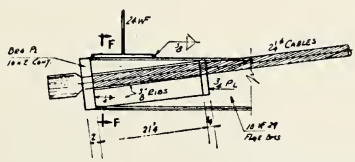
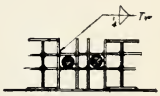












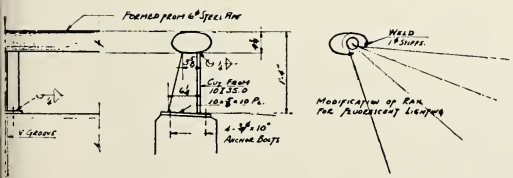


Figure 34

as the 528 ft. Frankenthal Bridge at Cologne (1939), the 606 ft. Cologne-Deutz Bridge (1948), and the 643 ft. span Bonn-Beul Bridge (1951), followed by the Dusseldorf-Neuss Bridge (1951) of 676 ft. span. It is indeed astonishing to realize that girder designs have been prepared for spans up to 1312 ft. [see F. Dischinger, 'Composite Steel Bridges with Slabs in Compression', *Bauingenieur,* Vol. 24 (1951) No. 11, pp. 321-332; No. 12, pp. 364-376, and F. Dischinger, 'Stability of Composite Bridge Slabs', *Bauingenieur,* Vol. 24 (1951) No. 4, pp. 103-106], and most recent reconstruction programs in Europe contain girder design proposals involving spans of 1000 ft. [see 'Competition for Reconstruction of Rhine Bridge at Cologne-Mullheim', Ed. by K. Schaechterle and W. Rein, Springer Publ., 1950]. . . . It is of interest in this connection that the European designs are invariably welded, while the American designs have invariably been riveted.

"In other words, girder type of bridge construction has been found practical in the range from ordinary short-span expressway overpasses to the comparatively long-span structures heretofore relegated to trusses, arches, or other structural types. This widespread adoption is in no small measure due to the relative simplicity and unobtrusive elegance which may be achieved with this type, especially in deck construction, in comparison with others which are less compact in their proportions. It is rather difficult, for example, to visualize other types satisfactorily replacing the ribbon-like laciness of the Hackensack-Passaic River Bridges, of the TVA Bridge over the Fontana Reservoir, of the viaduct of the Gowanus Elevated Parkway, or of the South Street Bridge in Washington.

"This entry integrates the following well-established means of securing economy of steel:

a. Welding.
b. Composite Construction.
c. Prestressed Steel.

The final end product in itself may not be well known, but its merits have been recognized and advocated by many outstanding authorities, as will be brought out presently.

"*Welding:* It is well known, and perhaps needs no elaboration, that structural welding in ordinary bridge structural work will achieve savings of from 10 to 20 per cent in tonnage merely by virtue of simplicity of design details (such as stiffeners), by the increase of net flange area, and by improvement in the effective depth of girders with the use of welding. Maximum overall savings in steel, however, can only be achieved when welding is correlated

with and can be adapted to auxiliary structural systems which in themselves are known to produce steel savings. Composite construction and prestressed steel construction are two of these which lend themselves exceptionally to such integration. Both of these essentially involve the substitution of other materials more effective and more economical than structural steel.

"*Composite Construction:* Composite construction implies the tying in of the bridge slab by suitable shear connectors to the steel girder, so that the entire section acts as a structural unit. In essence, this means that the concrete is utilized in compression (where it is most effective and economical), while the steel acts in tension (where it is most effective and economical). Composite design is recognized by most structural authorities as an effective means of insuring economy (particularly in steel tonnage), of promoting shallow depth and more graceful structural lines, and of improving the rigidity of bridges. Typical savings produced with composite construction alone are in the range of 8 to 30 per cent by weight of steel [see C. P. Siess, 'Composite Construction for I-Beam Bridges', *ASCE Transactions,* Vol. 114 (1949), pp. 1023-1045]. To be effective, of course, the concrete must always be in compression if cracks are to be avoided in the pavement. It is one of the features of a prestressed bridge that this requirement is automatically met by the technique prescribed below, thus eliminating the difficulties heretofore encountered in continuous composite bridges.

"*Prestressed Steel:* Prestressed steel is a new and at the same time very old type of construction, which has been recently advocated fairly extensively, notably, by Magnel [see G. Magnel, 'Prestressed Steel Structures', *The Structural Engineer,* Vol. 28, No. 11, Nov. 1950, pp. 285-295; July 1951, pp. 203-206. Also *Oss. Metallique,* Vol. 15 (1950) Nos. 6 & 9. Also Stahlbau, Vol. 20, No. 10, Oct. 1951, pp. 127-8], Dischinger, and Coff [see 'American Engineer Studies Prestressing of Structural Steel', *Civil Engineering,* Nov. 1950, p. 746]. Magnel has demonstrated quite simply, for example, that savings in weight of 50 per cent, and in cost of 20 per cent, are possible by prestressing simple steel tension members, and it is not difficult to extend his reasoning to apply to steel girders as well. The savings indeed depend on the ratio of dead to live load, and the more the dead load the higher the savings. It is for this reason that the method is particularly suited to the longer spans, although even on shorter spans it shows considerable savings.

"Professor Dischinger's modification of the above is to incorporate composite construction in such a way that the same cable system

which prestresses the steel, automatically precompresses the concrete slab, and in this way the common objection to the use of composite construction in continuous spans is removed completely. The resulting economy is much greater than that shown by Magnel, and by means of this Dischinger proposed spans up to 1312 ft. as economically feasible.

"It must be pointed out here that there is nothing basically new in these structural schemes, which have actually been known since the inception of the science of structural mechanics. The ideas behind the king-post and queen-post truss, or the trussed beam, are identical in principle with those described above. Furthermore, the construction is in a way nothing more than a self-anchored suspension bridge, concerning which we have ample and specific knowledge and experience. In such bridges, for example, the use of the deck slab as a composite construction to resist the horizontal component of cable stress, is fairly common, as noted by McCullough, [see C. B. McCullough, G. S. Paxson, and D. R. Smith, 'An Economic Analysis of Short-Span Suspension Bridges for Modern Highway Loadings', Tech. Bulletin #11, Oregon State Highway Dept., Jan. 1938, p. 37].

"While in theory the idea of prestressing as above outlined is basically the same as that employed in prestressed concrete, there are certain notable differences which make the present design far more advantageous in application. Particularly is this true of the fact that the prestressing cables are not imbedded in the concrete; therefore, the operation of prestressing is easily accomplished and the high labor costs associated with current prestressed concrete in this country do not appear here. Further, there is better control over creep and shrinkage effects on prestress, for which adjustments may easily be made at proper intervals. The large cables employed are essentially the system of 'Americanized prestress', in which the loading of multitudinous single wires is dispensed with.

"Under this system of prestressing, the aim essentially is to have the dead load carried as much as possible by the cables. If this condition is achieved perfectly, the cables induce in the girder moments exactly opposite to and counterbalancing the dead load moments, arriving at a so-called 'column condition' in which (for this design) the slab is in compression throughout. If the slab is free from the steel girder during the prestressing of the cables, then for the perfect column condition there would be no stresses of any kind in the steel girder due to dead load! The achievement of such a fine balance may not, however, be necessary or even desirable for

practical reasons, and the designer has complete latitude in his choice of prestress conditions and the degree to which the perfect column condition is achieved. After prestressing, the slab is rigidly connected to the girder, and thereafter the live loads act upon the composite section. It is noted here that such connection is ideally made by welding, while riveting, bolting and other media do not lend them selves readily to this application.

"The normal sag-span ratios for the cable are such that the live load is almost wholly carried by the composite girder section which is far stiffer than the suspension system. As in prestressed concrete, this fact results in a very much better factor of safety against overstress failure of the cable, and an allowable stress of 100,000 p.s.i. is considered permissible in the cable.

"The design is carried out, of course, so that sufficient compression is placed in the slab so that even under the worst negative live load moments no net tension (and therefore possible cracks) may occur in the slab.

"This Design: It was originally contemplated in this entry to incorporate the above principles in a redesign of the new big Hackensack-Passaic River girder spans, both of which are riveted. However, since the structural system would be different, such comparison would not meet the rules of this competition; nor would the ethics of such an attempt be easy to resolve among the many engineers responsible for the above bridges, which unquestionably are well conceived and represent the highest degree of structural design skill thus fare demonstrated in this country. Accordingly, the decision was made to prepare an independent set of design calculations (both welded and riveted) for a somewhat longer span length, but which would still be typical of the system described and would illustrate not only the advantage of welding vs. riveting, but also the features of steel conservation mentioned over and above the conventional types of girder construction.

"In essence this entry should be judged as a whole system of design, applicable to wide ranges of spans and conditions, as well as to other bridge types—such as suspention, arch, rigid frame and others, as will be described later,—rather than to the specific proportions of the design submitted, which are primarily for purposes of illustration. Some discussion of such extensions is given subsequently in this report.

"*Erection:* It must be clear from the previous discussion (see paragraph, 'Prestressed Steel') that truly phenomenal savings in structural steel are conceivable in a system where the cables and

slab carry all of the dead load. Indeed, if the bridge is supported on falsework throughout the various phases of erection and prestressing, the steel begins to assume a minor structural role. An analagous (though not identical) condition is well known to designers familiar with composite construction.

"In order to emphasize the versatility of the proposed design procedure, however, and to illustrate its application to the case where the girder is to be erected by the cantilever method, the entry submitted has gone beyond the above comparatively simple solution and has been so designed that no modification of section is necessary to allow such cantilever erection without overstressing the elements of the bridge.

"With the use of falsework, the steel girder serves during erection merely to carry the slab from bent to bent of the falsework, and the prestressing essentially lifts the girder from the falsework. In the cantilevering procedure the girder must in itself be capable of carrying the dead load over the entire span while the slab is being poured or is setting, and prior to slab prestressing. The amelioration of this latter predicament is acomplished in part by the use of auxiliary prestressing cables anchored to the steel, and in part by the higher stress values allowed when erection stresses are included.

"Not only is welding recognized as the 'natural' connecting device in any system incorporating composite construction, but the design technique introduced in this entry brings additional and important advantages with respect to welding. If the table of stresses and the stress curves on Fgiure 34 be studied, it will be apparent that the live load stress variations from the dead load stress condition (#4, #5a, #5b, #5c) are relatively low, and hence fatigue problems are eliminated in large measure. Furthermore, the cable suspension system reduces shears, greatly lowering weld requirements in the flange elements.. As a consequence of these facts, the normal amount of welding necessary from a practical standpoint to join the girder components is found more than ample from all stress standpoints.

"*Erection Sequence:* Assuming the side spans erected by conventional methods employing one or more temporary bents and cantilever erection, the major steps in the erection of the main span are as follows:

#1. Cantilever individual girders out 90 ft. from piers and hoist center section in place.

#2. Add auxiliary cables at supports and part of main span cables (W and X), all anchored to structural steel. Pre-

stress to full value.

#3. Pour slab on formwork resting on girder steel. Slab is independent of main structural steel. The slab is poured directly on the top ¾ in. plate with the shear connectors, this plate in turn resting on rollers on the main girder flange.

#4. After setting of slab, connect remainder of main span cables (Y) and prestress against slab. Temporary rollers still in place. If desired the construction may be allowed to stand a short period and readjustment of prestress made to take up shrinkage and plastic flow.

#5. Weld ¾ in. plate to girder flange by continuous side plates shown in the drawings. Henceforth structure acts as a composite under live load.

Various live loadings were investigated:

#5a. Max. positive moment at center of main span.

#5b. Max. negative moment over pier.

#5c. Live loads in end spans only.

"The stresses produced by the above loadings, including the various erection stresses, are plotted on Figure 34. . . . It must be noted that the stresses are combined stresses ($P/A + Mc/I$); therefore the conventional plots of maximum moments vs. moment capacity are meaningless for this type of construction, and as a result the stress plot must be employed.

"The initial phase of the erection sequence is predicted upon the construction of the approach spans on falsework bents, the work progressing from the shore to the piers. In this way the box girder section at the piers can be set directly in place by derricks, the sections being allowed to ride the completed portions of the approach span up to the derrick point. Such a sequence was used in the monumental Dusseldorf-Neuss Bridge erection [see K. Schaechterle and L. Wintergerst, 'Reconstruction of Rhine Bridge at Dusseldorf-Neuss', *Bauingenieur,* Vol. 27 (1952) No. 1, pp. 1-19], and avoids difficulties with field splicing of the deep plate girder sections if attempted from the pier outward; of course, the method is ideally suited to box girder sections, such as is employed in this design at that point.

"The analytical methods of prestressed steel [see Magnel reference], and of prestressed concrete are now so well known that it is not necessary here to elaborate on them. Typical design calculations are appended to this report so as to illustrate the procedures. Suffice it to say that methods similar to those recently expounded

by Moorman [see R. B. B. Moorman, 'Equivalent Load Method for Analyzing Prestressed Concrete Structures', *Journal ACI,* Vol. 23, Jan. 1952, No. 5, pp. 405-416], are all that are needed for the background for such calculations.

"Designers will undoubtedly find considerable favor in the system employed, because of the complete latitude given them in selecting prestressing forces and geometry to meet the physical loading conditions. Advantage is taken of this in the specific design, for example, to adopt the cables to a system of erection employing cantilever construction.

"It is a point of interest brought out in the calculations than an ultimate strength check indicates factors of safety against ultimate failure of 2.18 and 2.62 at the critical points of the span, both considerably above normally accepted values of 1.9 and 2.0. The use of such supplementary ultimate analysis has further practical value in that it may be employed as a means of preliminary design in selecting sections.

"*Comments on Details:* The shallowness of the steel construction (depth-span ratio 1:45) is worthy of note. This is by no means a limiting value, for the literature contains others with this structural system relatively much shallower.

"The slab concrete employed has an ultimate value of 5000 p.s.i. in 28 days. The production of concrete of this quality is fairly routine with modern methods of mix design and control, and with the increasing degree of practice with prestressed concrete requiring such quality.

"The elimination of floorbeams is noteworthy. The economic desirability of such a construction is attested by the adoption recently of a comparable system (without floor steel) to the Rio Paz Bridge between El Salvador and Guatemala by the John A. Roebling's Sons Co.

"The box-like structure—closed over the piers, and open at the centers of span—develops improved wind resistance. The bracing systems were incorporated mainly for erection conditions prior to the pouring of the slab, and of course lend themselves to further increasing the wind resistance of the structure after the above pouring period. Another advantage of the box-structure is a better distribution laterally of the live loading, an important factor in improving the ultimate failure strength of the structure.

"The parapet, railing, and sidewalk [see Figure 34] were specifically adopted from the New Jersey Turnpike structures, as representing modern examples of good practice in this field. The

addition of fluorescent lighting in the handrails represents a modern improvement in detail used with success on some bridges in this country, and eliminating the unsightly overhead light standards.

"The flange makeup in the welded design, using 8 x 8 x 1 angles, has been employed in many welded bridges built in the past. It has the specific advantage of improved stress flow to the flange plate, which in this instance must be fairly wide. Conventional riveted construction (see Figure 33) is inferior in this respect. Further, the use of the welded flange reduces the depth of the web in respect to buckling.

"Webs are stiffened at piers both horizontally and vertically. Erection of the entire center pier section on the ground at the bridge site is considerably simpler than the building up of the box section directly on the pier. This is attested to by the adoption of such a scheme in the long-span Dusseldorf-Neuss bridge.

"Stringing of cables and prestressing are accomplished in stages as required by the erection stress conditions. . . . Cables of 2¼ in. and 3 in. diameter are standard galvanized bridge cables, as used in suspension and other construction. Certainly the danger of corrosion is negligible with such tested materials. Cables are of course free for inspection and maintenance as required.

"True composite construction does not take place in this system until the ¾ in. plate under the slab is connected to the top steel flange. The achievement of this connection is certainly one place where welding is so far superior to riveting that riveting is out of the question. The riveted junction would require at least field reaming—if not field drilling of the holes, which would hardly be desirable to the extent that it would be necessary.

"Likewise, the shear connection between the slab itself and the ¾ in. plate on which it rests is an item of construction in which welding is ideal; the welded detail has so far a superiority over the riveted one that it is questioned whether much support could be found for the latter.

"The live load deflection is low with such structural systems employing cables, and amounts to 1/900th of the span for the design indicated. Dead load deflection is zero by virtue of the lifting of the cables.

"Shears are particularly low. The ordinary minimum size of welds required for practical connections constitute more than ample allowance for the stresses involved. Web plates are also comparatively low-stressed.

"In order to give a valid comparison, the riveted sections were

so designed as to yield the same stresses as attained on the welded design. Riveted sections are governed by the specification requirements governing net section. The riveted design is further penalized by less flexibility with relation to the design of stiffeners, etc., which must be in the form of angles for proper connection.

"An important advantage of the use of cables is that adjustment of their stress or of their number is possible with future changes in live loadings, thus increasing the life and utility of the bridge.

Quantities:

"Quantities for both designs are summarized below. For details, see the calculation sheets.

	Exhibit A Riveted	Exhibit B Welded
Concrete, cu. yds.	1,840	1,840
Reinf. steel, lbs.	383,700	383,700
Galv. Bridge Cables, lbs.	382,000	382,000
Cable Fittings, lbs.	8,640	8,640
Structural Steel A-7, lbs. (incl. 67,000 lbs.) (c.s. saddles)	4,280,900	3,554,100
Structural Steel unit wt.		
lb./sq.ft. roadway	120.5	99.5
lb./sq.ft. bridge plan	98.8	81.8
Weld Weight lbs.		Shop 21,200
		Field 8,429

"The riveted design thus uses 21 per cent more structural steel than does the welded bridge.

"The following discussion will further elaborate on the steel savings of this particular design and the technique in general. It is well known that bridges in this span classification generally utilize from 100 to 150 lbs. of high strength steel per square foot of roadway surface. The Hackensack Girder (silicon steel) falls within this range (as judged from the reaction and sheer data published in a recent article, see New Jersey Turnpike reference). The recent long-span girder at Dusseldorf-Neuss, over the Rhine, of box construction throughout, used 515 kg./m^2 = 106 lbs. per square foot of bridge of high strength steel, which compares with the figure of 81.8 lbs. per square foot of bridge plan of this design, using ordinary structural steel. (If cables and fittings are added to the latter figure, it becomes 92.7).

"Seeking further comparisons, it may be noted that a typical formula [see Huette Handbook, Vol. III, p. 344], for highway

bridge weights in this class is (structural steel)

$W = 10.2 - .123\ L - .00019\ L^2 -$ (31 to 51 for roadway steel) lbs. per square foot of bridge plan.

For the design in question, this is

$W = 10.2 - .123 \times 450 - .00019 \times 450^2 -$ avg. 41 = 145 psf bridge plan of structural steel.

The design submitted achieves the same thing with 81.8 or 92.7 psf, depending on whether the cables are counted in or not.

"A further comparison is given by the competition preceding the Cologne-Mullheim bridge construction which produced designs ranging from 0.5 to 0.9 t/m (= 103 to 180 psf of bridge plan) for girder construction, all using *high strength steel.*

"The above demonstrates that the given design not only uses 21 per cent less steel than the riveted analog, but that the structural system adds savings several times more than this in comparison with standard constructions, most of which had to resort to high strength steel to keep within reasonable size limits.

Costs:

"It is not felt that a detailed analysis of costs of such a system will reveal much difference in the per pound price between riveted and welded steel. This observation confirms general observations by others in recent literature, and is more applicable to the present design in which any difficulties which would arise in cost analysis would be associated with the conditions of erection and the familiarity of the contractor with the system of prestressing.

"Using typical in place costs of 17c per lb. for structural steel, 11c for reinforcing steel, $100/yd. for the high-strength concrete (which it is felt is perhaps more liberal than necessary), 50c per lb. for the cables, we arrive at a cost of the bridge superstructure of:

	Riveted	Welded
Concrete	$ 184,000	$ 184,000
Reinforcing	42,200	42,200
Cables	191,000	191,000
Fittings	4,300	4,300
Structural Steel	727,800	604,200
Total	$1,149,300	$1,025,700

Thus, the weight savings carries over directly into cost savings, although not in the same ratio, since the extra cost of concrete is high relatively."

CHAPTER III

SIMPLE BEAM SPANS

In comparison to the number of continuous span bridges presented, the simple span structures were relatively few; however, there were enough to represent various types of construction. Some are curved, some straight and some are skewed and others have square ends. Normal reinforced concrete slabs are used for the roadway, others have composite slabs, and still others have battle-deck floors.

Henry F. Gauss, Moscow, Idaho, designed a welded bridge of three simple spans—approximately 32 ft., 78 ft., and 32 ft. respectively. It is the counterpart of a riveted structure already built which is on a skew and a 3° curve. The bridge carries two lanes of roadway (24 ft.) plus one sidewalk on four lines of beams with the reinforced concrete slab spanning the 7 ft.-6 in. between beams since stringers and floorbeams are not required. The plan view of Figure 35 shows the cross frame spacing.

The riveted beams are 24 WF 76 for the shorter end spans and fabricated girders using 48 in. by ⅜ in. webs, 6 x 4 x ⅝ in. flange angles, and 14 in. cover plates for the center span. The stiffeners are 4 x 3 x 5/16 in. angles spaced at about 3½ ft. centers.

The welded beams for the end spans consist of 26 x ⅝ in. webs and 9 in. channels as flanges. For the center spans, the web is a 50 x 5/16 in. plate with 4 x 5/16 in. plates as stiffeners and the flanges are 12 in. ship channels (see Figure 36). The manner in which the channel flanges serve as the forms for the haunches in the concrete slab is shown in the cross sections of Figure 37.

Because the bridge is on a curve and there is a sidewalk on only one side, the girder nearest the sidewalk (A4) is strongest and the girder next to it (B4) has the least strength. The two girders away from the sidewalk (A3 and B3) are of equal size. A comparison of the weights of these girders both riveted and welded is thus:

Girder	Weight-Riveted	Weight-Welded	Per Cent Savings
A4	16759	12679	24.4
B4	14897	11330	24.0
A3	15183	11850	21.9
B3	15183	11850	21.9

The total weights for Exhibit A are 102,844 lb. of structural steel plus 4,000 lb. of rivets. For Exhibit B, the structural steel totals 82,880 lb. and the welding rod amounts to 1,402 lb.

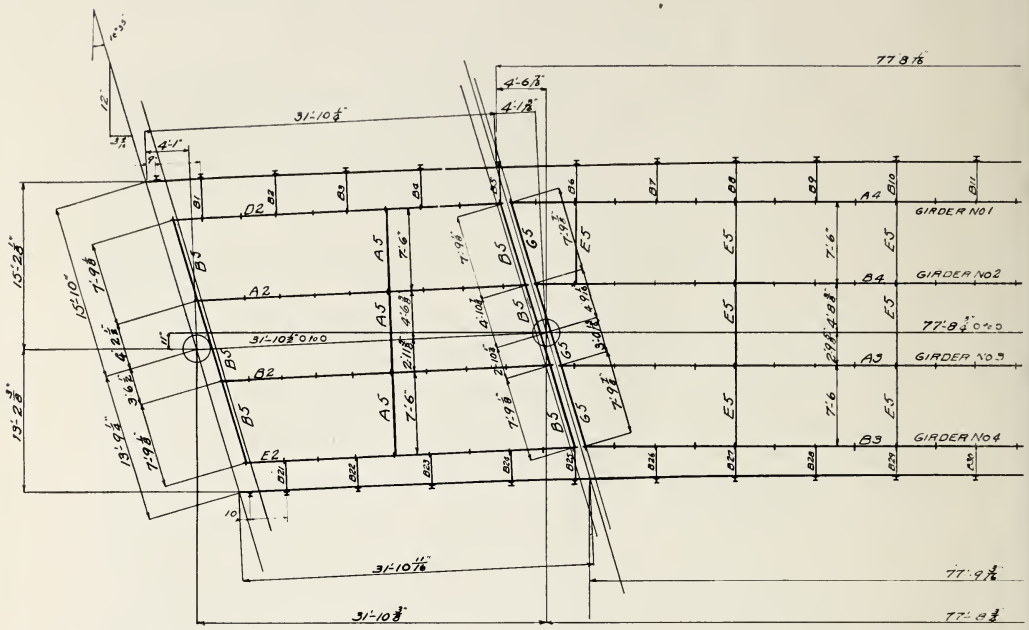
After summarizing his weights and costs, Mr. Gauss said: "The difference in cost is $6040.46 which represents a saving of 30 per cent. It must be remembered that the figure represents the saving in cost of structural steel work only; when the cost of the abutments, piers, road-way and grading is included the above saving forms a much smaller per cent of the total."

J. D. Jewell, Oklahoma City, Oklahoma, utilized three simple spans for his skewed bridge crossing. The span lengths are 67 ft., 65 ft., and 67 ft. The total width of 74 ft.-8 in. provides for two 27 ft. roadways and a 15 ft. median strip (see Figure 38).

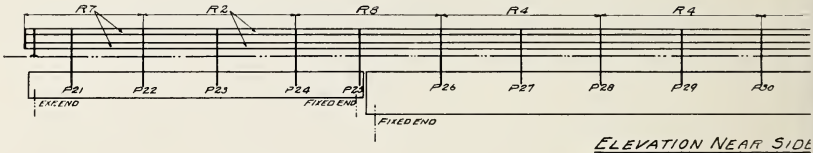
The reinforced concrete slab is designed to act compositely with the longitudinal beams. These beams are 27 WF 94 sections that have been split longitudinally and welded into beams of variable depth as shown in Figure 39. At the ends the beams are 12 in. deep, and for the middle 15 ft., the beams have a constant depth of 32 in.

The composite slab is supported directly on the main beams which have a spacing of 7 ft.-6 in., except at the median where one space of 7 ft. even is used. The slab has a 6 in. haunch at each beam. The bridges are separated along their centerlines into two halves with no connection between the halves.

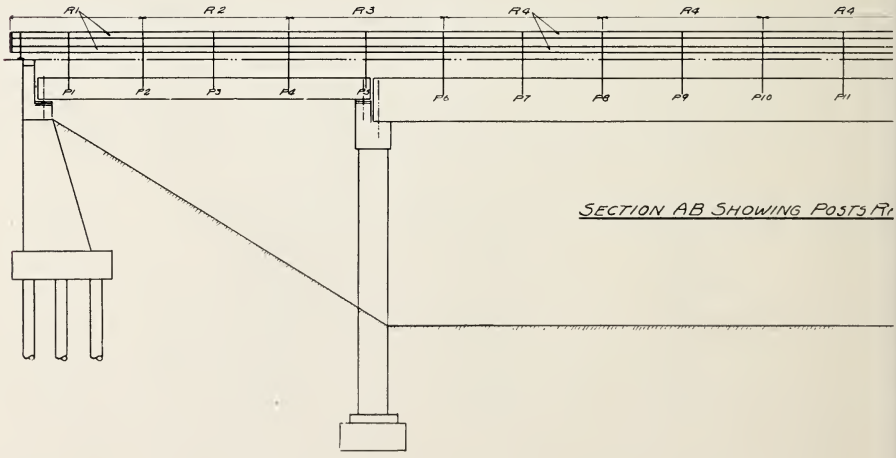
The welded bridge requires 446 cu. yds. of concrete, 96,490 lbs. of reinforcement steel, 213,055 lbs. of structural steel, 2,738 lbs. of shop welds, and 95 lbs. of field welds.



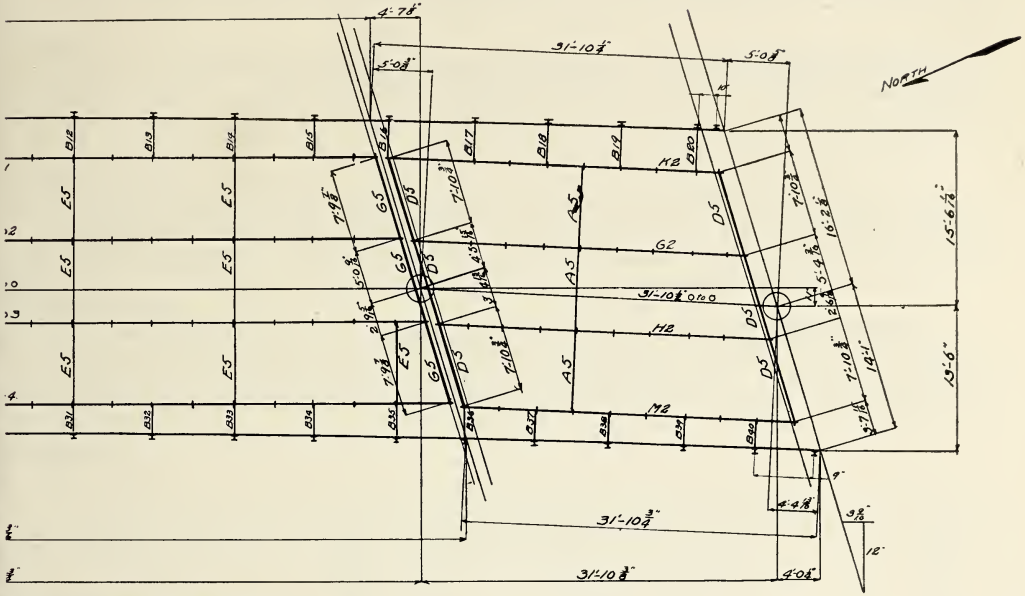
PLAN OF GIRDERS (



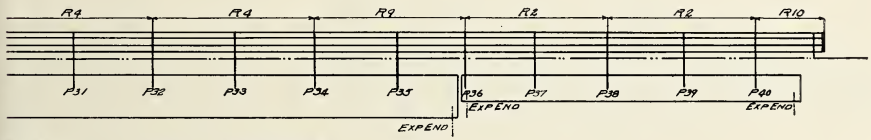
ELEVATION NEAR SIDE



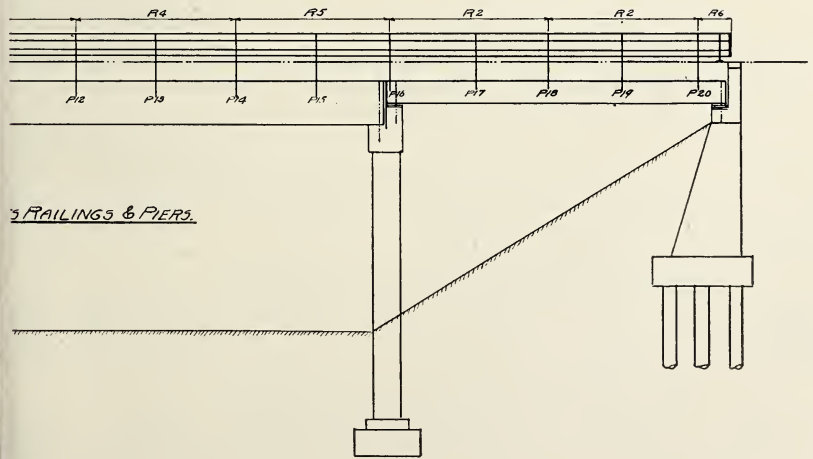
SECTION AB SHOWING POSTS P1



RS CROSS FRAMES & BRACKTS

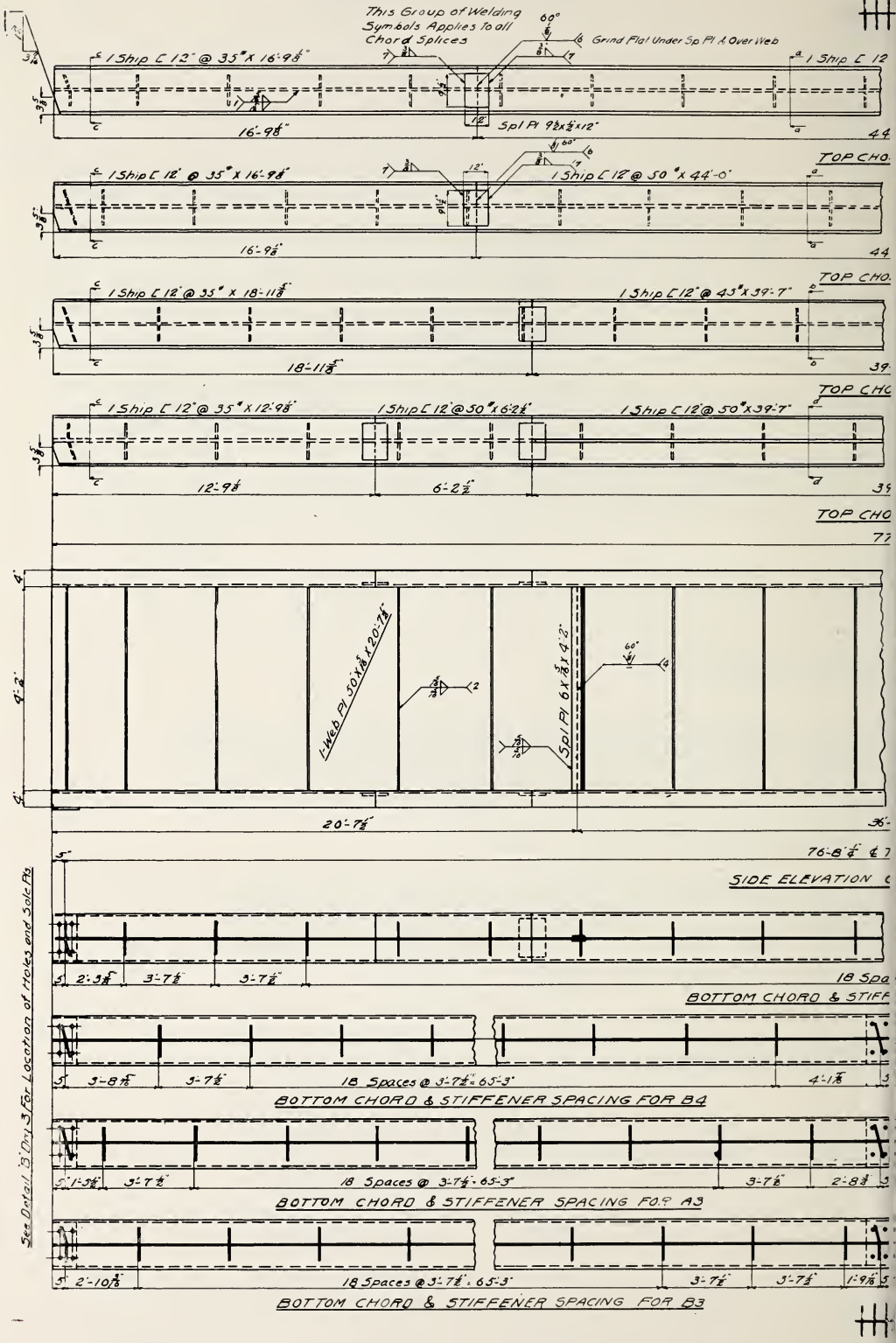


SIDE SHOWING POSTS AND RAILING



S RAILINGS & PIERS.

Figure 35


This Group of Welding Symbols Applies to all Chord Splices
Grind Flat Under Sp Pl & Over Web
1 Ship C 12" @ 35" X 16'-9⅞"
16'-9⅞"
Spl Pl 9⅞"x⅞"x12"
1 Ship C 12" @ 50" X 44'-0"
TOP CHO
1 Ship C 12" @ 35" X 18'-11⅞"
1 Ship C 12" @ 43" X 39'-7"
1 Ship C 12" @ 35" X 12'-9⅞"
1 Ship C 12" @ 50" X 6'-2½"
1 Ship C 12" @ 50" X 39'-7"
12'-9⅞"
6'-2½"
TOP CHO
1-Web Pl 30"X⅜"X 20'-7⅞"
Spl Pl 6"x⅜"x 4'-2"
4'-2"
20'-7⅞"
76'-8"
SIDE ELEVATION
2'-3⅜"
3'-7½"
3'-7½"
18 Spa
BOTTOM CHORD & STIFF
3'-8⅞"
3'-7½"
18 Spaces @ 3'-7½" = 65'-3"
4'-1⅞"
BOTTOM CHORD & STIFFENER SPACING FOR B4
1'-3½"
3'-7½"
18 Spaces @ 3'-7½" = 65'-3"
3'-7½"
2'-8⅜"
BOTTOM CHORD & STIFFENER SPACING FOR A3
2'-10⅞"
18 Spaces @ 3'-7½" = 65'-3"
3'-7½"
3'-7½"
1'-9⅛"
BOTTOM CHORD & STIFFENER SPACING FOR B3
See Detail '3' Dwg 3 For Location of Holes and Sole Pls

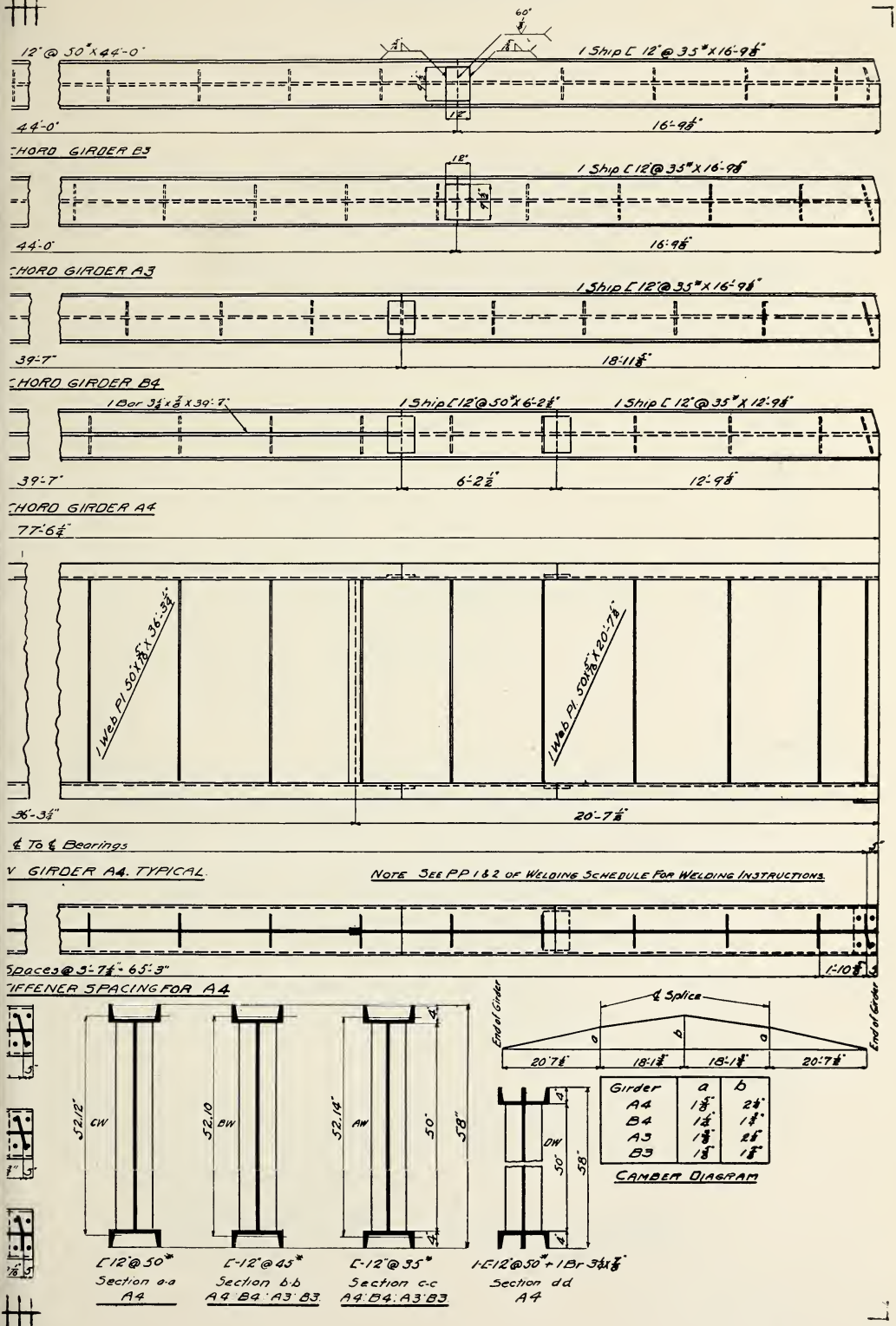
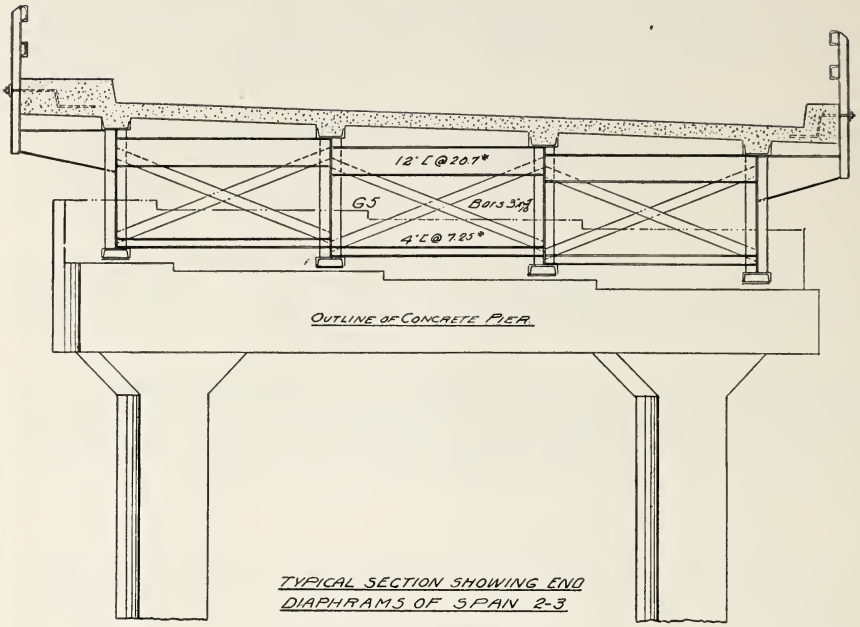
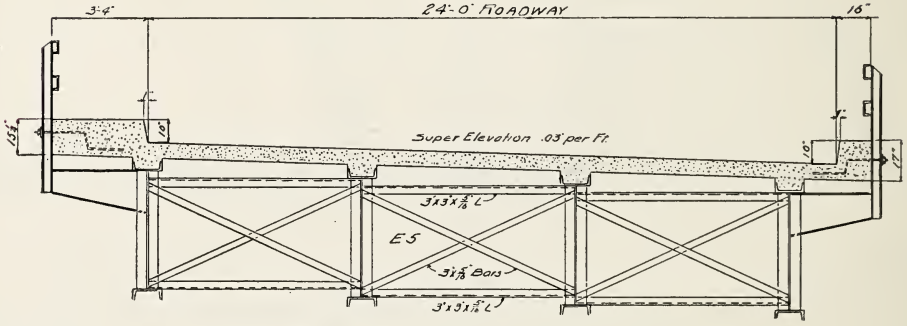

12" @ 30"X44'-0"
1 Ship L 12"@35"X16'-9⅝"
44'-0"
16'-9⅝"
CHORD GIRDER B3
1 Ship L 12@35"X16'-9⅝"
44'-0"
16'-9⅝"
CHORD GIRDER A3
1 Ship L 12"@35"X16'-9⅝"
39'-7"
18'-11⅝"
CHORD GIRDER B4
1 Bar 3½"x⅞"X39'-7"
1 Ship L 12"@30"X6'-2½"
1 Ship L 12"@35"X12'-9⅝"
39'-7"
6'-2½"
12'-9⅝"
CHORD GIRDER A4
77'-6¼"
1 Web Pl 30"x⅞"x 36'-3½"
1 Web Pl 30"x⅞"x 20'-7⅛"
36'-3½"
20'-7⅛"
℄ To ℄ Bearings
V GIRDER A4. TYPICAL
NOTE SEE PP 1 & 2 OF WELDING SCHEDULE FOR WELDING INSTRUCTIONS.
Spaces @ 3'-7½" = 65'-3"
1'-10⅝"
STIFFENER SPACING FOR A4
52.18"
CW
52.10
BW
52.14"
AW
50"
58"
DW
℄ Splice
End of Girder
End of Girder
a
b
a
20'-7⅛"
18'-1⅝"
18'-1⅝"
20'-7⅛"
Girder | a | b
A4 | 1⅜" | 2½"
B4 | 1⅜" | 1⅜"
A3 | 1⅜" | 2½"
B3 | 1⅜" | 1⅜"
CAMBER DIAGRAM
L 12"@30"
Section a-a
A4
L-12"@45"
Section b-b
A4 B4 A3 B3
L-12"@35"
Section c-c
A4 B4 A3 B3
1-L 12"@30" + 1 Bar 3½"x⅞"
Section d-d
A4

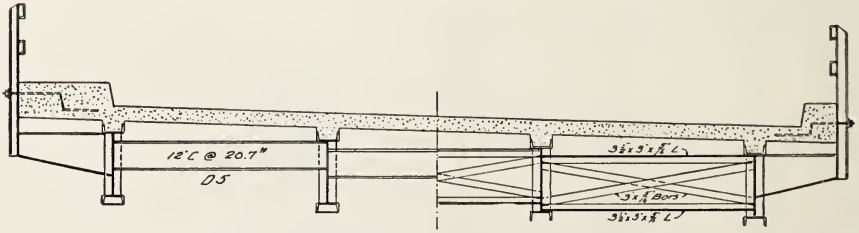
Figure 36



TYPICAL SECTION SHOWING END DIAPHRAMS OF SPAN 2-3

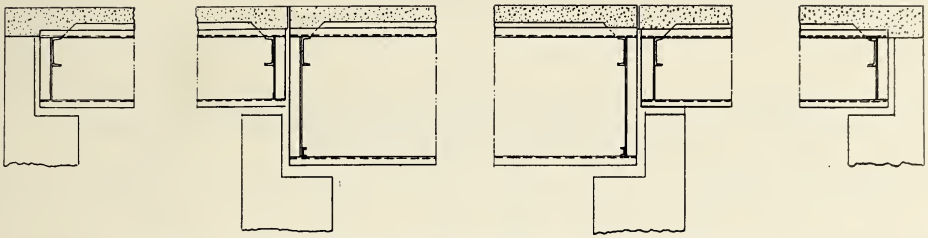


TYPICAL SECTION SHOWING INTERMEDIATE DIAPHRAMS OF SPAN 2-3

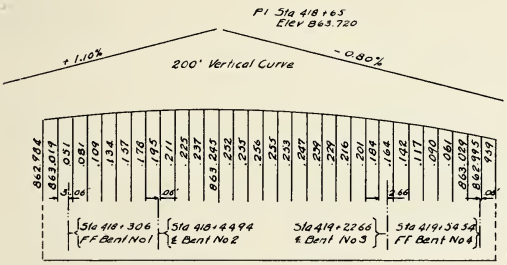


HALF SECTION SHOWIING END DIAPHRAM OF END SPANS. 1-2 AND 3-4

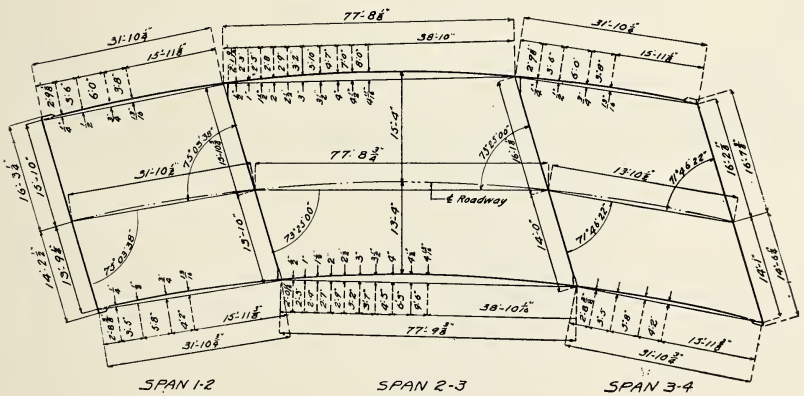
HALF SECTION SHOWING INTERMEDIATE DIAPHRAMS OF END SPANS 1-2 AND 3-4



PART LONGITUDINAL SECTION SHOWING GIRDER ENDS - PIERS - HAUNCHES.



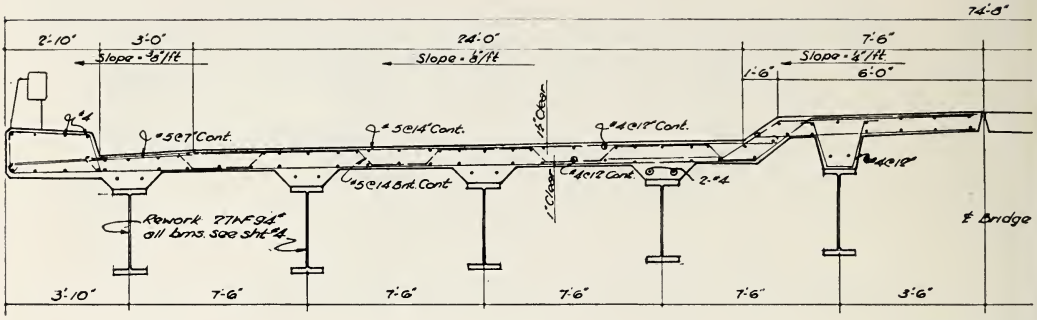
GRADE ELEVATIONS AT ℄ ROADWAY



PLAN AND DIMENSIONS OF FLOOR SLAB

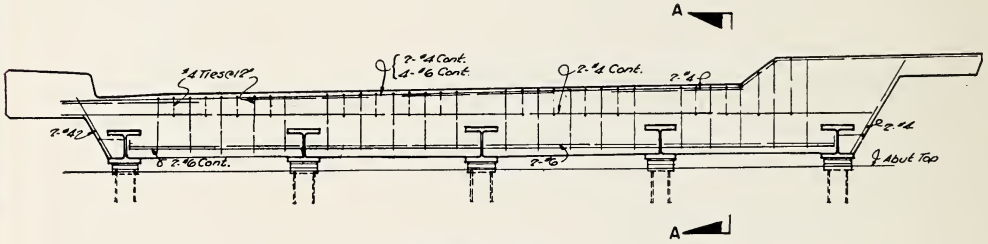
Weight of Reinforced Roadway Complete With Sidewalk, Curbs, and Haunches is 2930 Pounds per Lineal Foot of Bridge

Figure 37



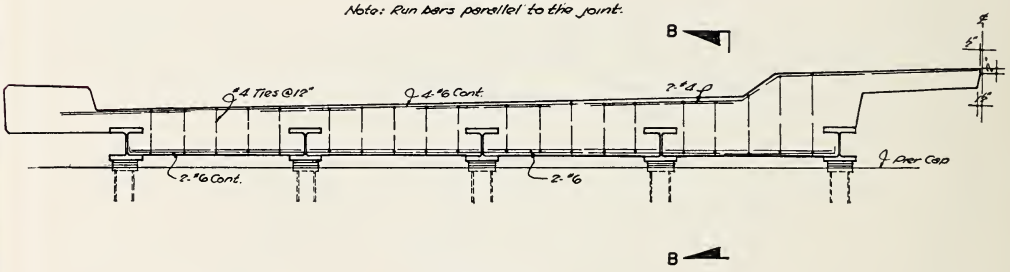
CROSS SECTIC

Cut @ right angles t
@ ℄ span



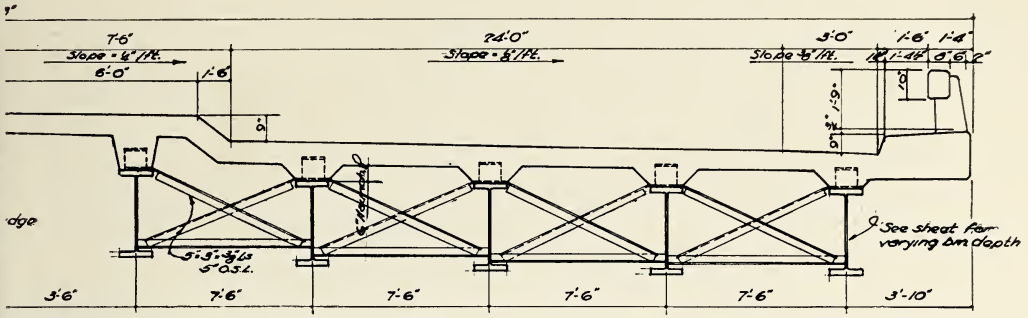
ABUTMENT DETAIL

Note: Run bars parallel to the joint.



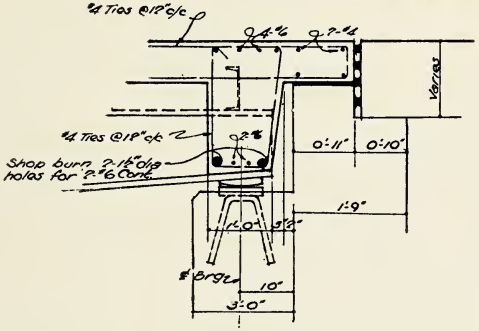
PIER DETAIL

Note: Run bars parallel to the joint

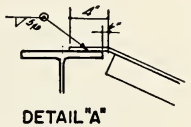


CTION
as to the beams
oan

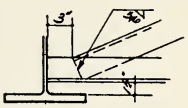
Note:
See diaphragm connection details "A & B"
Space 1'-0" ea side of ℄ of span. Place diaphrams @ right angles to beams.



SECTION AA

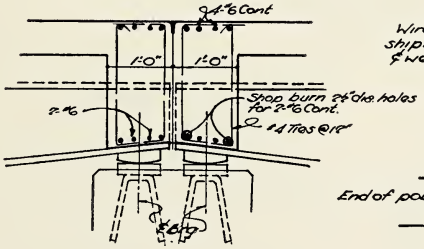


DETAIL "A"

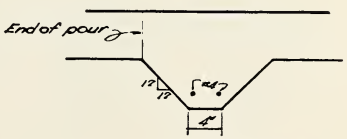


DETAIL "B"

Wire angles at the center for shipment. Adjust in the field & weld.



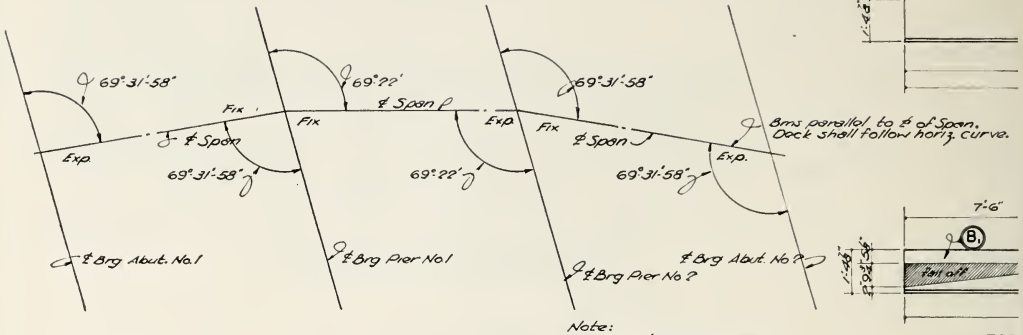
SECTION BB



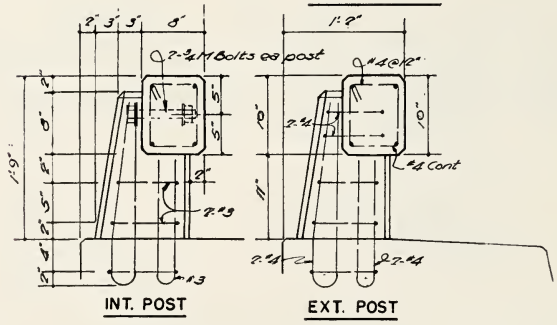
POUR DETAIL

If the slab pour is ceased the haunch @ right angles to the span must be used.

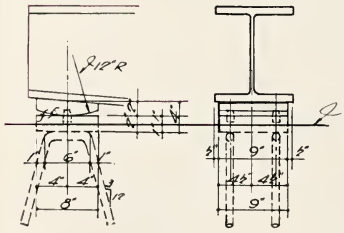
Figure 38



SPAN LAYOUT



HANDRAIL DETAIL

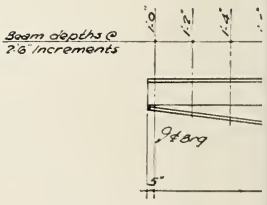


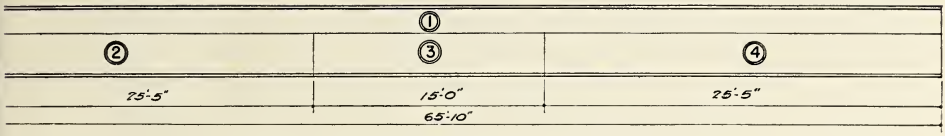
MATERIAL FOR BEARINGS

2-1B 8"x2'-0"-9"
2-1ϕ Bars x 2'-6"
2-1ϕ Pintles x 0'-1½" (fix shoes only)

Note:
Base brg plates shall be set when top of piers & abuts. are being poured and adjusted to grade.

BEARING DETAIL

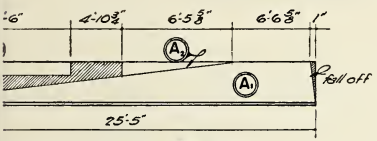




TORCH CUTTING OF 27 WF 94

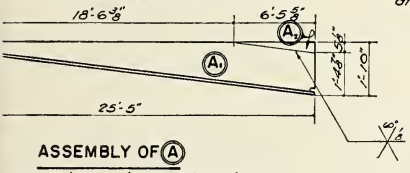
Torch cut four sections as noted above. Use multiple head cutting torch on track. Resulting beveled edges shall be ground smooth.

-ve.



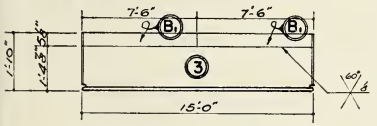
TORCH CUTTING OF ② & ④

ch cut sections as noted above



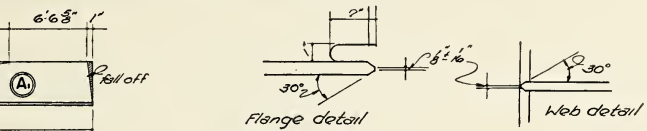
ASSEMBLY OF (A)

emble & weld torch cut sections noted above.



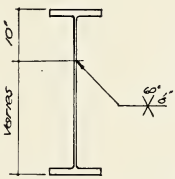
ASSEMBLY OF (B)

Assemble & weld torch cut sections as noted above.



EDGE DETAIL

Grind edges smooth after torch cutting.



BM. SECT.

WELDING PROCEDURE

Near Side Welder

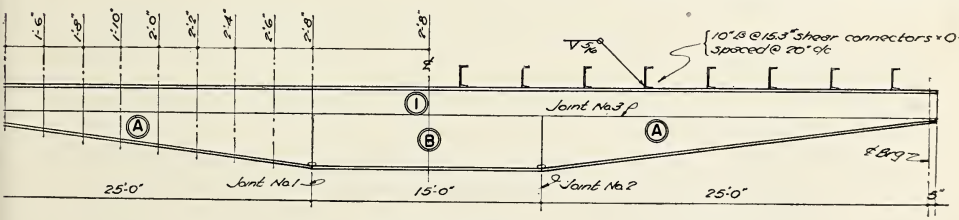
1. Weld bottom of bottom flange @ joint 1 (2 passes.
2. Weld top of bottom flange (N.S.) @ joint 1 (1st pass)
3. Weld top of bottom flange (N.S.) @ joint 2 (1st pass)
4. Weld web @ joint 2 (N.S.)
5. Weld top of bottom flange (N.S.) @ joint 2 (2nd pass.
6. Weld top of bottom flange (N.S.) @ joint 1 (2nd pass)
7. Weld web @ joint 1 (N.S.), plug weld slots.

Far Side Welder

1. Weld bottom of bottom flange @ joint 2 (2 passes
2. Weld top of bottom flange (F.S.) @ joint 2 (1st pass.)
3. Weld top of bottom flange (F.S.) @ joint 1 (1st pass)
4. Weld web @ joint 1 (F.S.)
5. Weld top of bottom flange (F.S.) @ joint 1 (2nd pass.
6. Weld top of bottom flange (F.S.) @ joint 2 (2nd pass)
7. Weld web @ joint 2 (F.S.), plug weld slots.

Notes

Both welders shall complete each step together. Joint no. 3 shall be welded with submerged arc automatic equipment. Joints nos 1 & 2 may be welded with automatic equipment upon approval of the procedure. The speed of travel, voltage, amperage, and rod size for the automatic weld on joint no 3, and joints no. 1 & 2 if they are welded automatically, shall be submitted for approval, and that procedure shall be used at all times.



FINAL ASSEMBLY

Assemble sect. 1 from torch cut bm. & sections A & B. Follow welding procedure as noted above.

Figure 39

CHAPTER IV

SIMPLE TRUSSES

Somewhat more than twenty-five per cent of all of the entries in this program are simple truss bridges. With but few exceptions, these structures are deck trusses. Some of the designs use the top chords as stringers, some have continuous stringers between trusses, and some have concrete slabs that act integrally with stringers, truss chords, or both. A few of these bridges have no stringers or floor-beams—the slab is supported directly on the top chords. Space trusses that eliminate the need of lateral bracing are utilized in one entry. Other variations involving the type of roadway and arrangement of trusses and their members were also presented.

James H. Jennison, Pasadena, California, presented a very complete design of a 150 ft. truss bridge. The end panels are 15 ft. long and the six intermediate panels are 20 ft. The truss has a curved bottom chord with a maximum depth of 15 ft. The 26 ft. roadway consists of a 7½ in. reinforced concrete slab that is designed compositely with the truss top chords (22 ft. apart) and three intermediate lines of continuous stringers.

The following is quoted from Mr. Jennison's discussion.

"Success in designing a bridge to conserve steel depends upon the judicious application of several design principles and careful design of connection details. The following principles were applied to conserve steel in the bridge designs presented herewith:

(a) The type of bridge should be selected to economize material for the particular span.

(b) Sections should place the material as far from the center as possible for the cross section area required. This results in large section modulus for beams, large resisting moment for trusses, and large radius of gyration for compression members.

(c) Loads should be carried to the abutments by the shortest possible path.

(d) Members should serve for more than one function, where possible, rather than individual members serving for each individual function.

(e) Beams should be designed continuous over supports where possible.

(f) The deck should participate with other members in carrying load to the greatest extent possible.

"A span of 150 ft. was selected, and a riveted bridge (Exhibit A) and a welded bridge (Exhibit B) were designed with similar features for comparison. Deck trusses of the Warren type were chosen as the type most likely to conserve steel for a bridge of this span. Moreover, a deck-type bridge has the additional advantages of affording an unobstructed view from the roadway, unlimited clearance above the roadway, and protection of the steel work from the weather and from damage by vehicles.

"Full advantage of the concrete deck in carrying loads to conserve steel was achieved by making the stringers continuous composite beams, by using the deck as a horizontal diaphragm to carry lateral loads, and by full participation of the deck and stringers with the truss top chords in carrying the compression load. Many bridges have been built with a concrete deck keyed to steel beams to produce a compositite member, and large savings of steel have resulted. However, as far as the author has been able to discover, deck truss bridges have never been built with the concrete deck carrying a large portion of the compression load. Such composite action between the deck and the trusses conserves a considerable weight of steel, particularly in a welded bridge.

"The welded bridge designed as Exhibit B with the deck participating with the stringers and top chords in carrying bending and compression loads is shown in Figure 40. Welding a bridge of this type makes possible a large saving in weight of steel and permits the elimination of much of the small detail material such as clip angles, gusset plates, stay plates, and lacing bars. The resulting design combines the advantages of steel conservation, low construction cost, durability, clean appearance, and low maintenance cost.

* * *

"The trusses were made 15 ft. deep for a span of 150 ft. giving a depth to span ratio of 1:10. This is the lower boundary of depth from the standpoint of the specification for limiting deflection. Neverthless, the weight of steel should be near the minimum possible, and the live load deflection should be smaller than the deflection experienced in a conventional truss because of the low top chord stress resulting from participation of the deck. The trusses can be transported to the construction site in three sections on low-bed trailers, whereas deeper trusses would require field assembly of individual members.

"A Warren type truss was selected because it gave a shorter stress

path to the abutments and permitted lighter verticals than the Pratt type. The horizontal shear from the diagonals is transferred to the deck at U_0, U_2, and U_4 (5 panel points) whereas a Pratt truss would have required that shear be transferred at all 9 panel points. The lower chord was curved to give more uniform size to the lower chord section and to the diagonals.

"Loads were applied at the panel points of both chords to determine the stresses in the truss members.

"Connections between truss members were made by means of gusset plates in the riveted bridge. However, in the welded bridge, the members were welded directly to each other with stiffeners provided as required to maintain uniform distribution of stress on the cross section of the member.

"The design of the top chord was tedious because several successive trial designs were analyzed before sufficient refinement was attained. Moreover, the stresses at mid-panel and at the panel points were necessarily analyzed separately because the bending moments, cross-sections, and in some instances the direct stresses were different. At mid-panel the full concrete area is always in compression, so the member can be treated as a composite beam subject to bending and direct compressive load. At the panel points the analysis is similar except that the upper part of the concrete is in tension and therefore is neglected. This required that the neutral axis be determined by trial, but by analyzing the errors it was possible to locate the neutral axis in close agreement with the stresses obtained from the usual elastic theory as applied to reinforced concrete. Formulas for stresses in the composite beams could be derived in the same way as the formulas for reinforced concrete beams subject to bending and compression are derived. However, a little investigation will show that these formulas are too complicated to be useful and that the trial method is best.

"A complete analysis of any member composed of more than one material should include the effects of temperature expansion and contraction and shrinkage. The thermal coefficient of expansion of concrete is close enough to that of steel that thermal effects can be neglected, as is usually done in reinforced concrete design. Shrinkage effects deserve more detailed consideration.

"Shrinkage, or shortening, of a concrete member arises from initial hardening of the concrete, from drying, and from plastic flow, or creep, under sustained load, and from elastic compression. The values of these factors that may prevail in the deck slab of the composite bridge are as follows:

	in. per in.
Shrinkage from initial hardening: (1)	0.0001
Shrinkage from drying: (2)	0.0008
Plastic flow or creep: (3, 4)	0.0004
Elastic compression:	0.0001
Total	0.0014

References:

(1) A. A. Anderson, "Expansion Joint Practice in Highway Construction", *Transactions* of the American Society of Civil Engineers, 1949, Vol. 114, p. 1162.

(2) Davis, ASTM *Proceedings,* Vol. 30, Part I, p. 677.

(3) R. E. Davis, H. E. Davis, and J. S. Hamilton, "Plastic Flow of Concrete Under Sustained Stress", *Proceedings* ASTM, Vol. 34, Part II, p. 34.

(4) J. R. Shank, "The Mechanics of Plastic Flow in Concrete", *Proceedings* of the American Concrete Institute, Vol. XXXII (1936), pp. 149-180.

"It is clearly evident from the above values of shrinkage and shortening effects that the steel beams will be stressed beyond the yield point in compression. However, the same theoretical conclusion applies to composite beam bridges of the usual type where a concrete deck is keyed to steel beams. Numerous bridges of this type have been built and have performed satisfactorily. Moreover, tests have indicated that the practice of designing the beams as elastic composite members without special consideration of shrinkage and plastic flow is satisfactory, [see C. P. Siess, 'Composite Construction for I-Beam Bridges', *Transactions* of the American Society of Civil Engineers, 1949, Vol. 114, pp. 1023-1072]. Furthermore, shrinkage and plastic flow in reinforced concrete columns frequently stresses the longitudinal reinforcement beyond the yield point but does not lessen the load carrying capacity as shown by numerous tests and much practical experience, [see F. E. Turneaure and E. R. Maurer, *Principles of Reinforced Concrete Construction,* 1935, Wiley, pp. 19, 185-193].

"Because of the above considerations the effects of shrinkage and plastic flow were not considered in the stress analysis. The bending stresses from dead loads were obtained by considering the steel beams acting alone; bending stresses from live load and direct stress were calculated on the basis of composite beam action based on the usual assumptions for reinforced concrete, namely: (1) Plane sections remain plane, and, (2) Hooke's law, stress is proportional

to strain.

"However, it seemed best to make the stringers and chords continuous so that the tendency for large strains to develop at local points would be minimized. The welded design, in effect, produces a single continuous member. The chords of the riveted bridge run through the joints and the abutting ends of chords and stringers are to be milled for bearing.

"Shear lugs made from a Z section were used on the riveted bridge, and shear lugs made from a channel section were used on the welded bridge. They were designed from test results, [see N. M. Newmark, F. E. Richart, and C. P. Siess, 'Discussion on Bridge Floors', *Transactions* of the American Society of Civil Engineers, 1949, Vol. 114, p. 1071]. Both the channel and the Z section tie down the deck in a positive way.

"The shear transfer at U_0 is accomplished by direct bearing on the end of the deck slab. At U_2 a special shear transfer device is provided. The hooked reinforcing bars carry most of the load, and no satisfactory substitute could be found for the riveted bridge. Since the bars could not be attached by any method except welding, they were welded on both bridges.

"The rivet heads no doubt provide considerable shear resistance on the chords of the riveted bridge; however, this was considered an added factor of safety rather than a primary load carrying factor. In this respect the riveted design appears to have an advantage.

"The bottom chord of the welded bridge is butt welded, taking full advantage of the gross section of the wide flange sections. The riveted bridge has a bottom chord composed of two channels in the conventional manner.

"The end connections of the tension web members of the welded bridge were designed with butt welds to avoid kinking of the stress paths. However, it was not feasible to avoid stress raising effects from abrupt intersection with other members. Consequently, to be conservative, the formulas from Table 1 of the AWS Specification for members end-connected by fillet or plug welds were used for all the web members and U_0 - L_1. The truck loading which governed these members was Category B, not more than two panels of loaded single lane, with a basic tensile stress of 14,000 psi. Two lanes loaded with a basic stress of 18,000 psi was a less severe condition for these members.

"The verticals of the welded bridge are made from pipe. Because pipe provides the largest possible radius of gyration in all directions for a given weight it is ideal for compression members. The welded

connections at the ends are simple and effective. Some eccentricity at the weld is not serious because of the polar symmetry of the pipe and its circumferential strength.

"The welded diagonals L_3 - U_4 are built up members composed of two channels and a bar to give a larger radius of gyration than can be obtained in a single rolled section of equal weight. Pipe was considered for the diagonals, but did not appear to be well adapted to joining the chords, particularly at the upper chord.

"The riveted bridge was designed with 8 WF 31 verticals and two channels for each diagonal. An alternate design using the following members was considered:

Verticals	10 WF 33
L_1 - U_2	10 WF 60
U_2 - L_3	10 WF 54
L_3 - U_4	10 WF 49

In many respects the wide flange diagonals are more satisfactory, and most designers would no doubt choose them in preference to the channels. However, the heavier main material, wider chords, and the fillers increase the weight by 4,075 lb. more than the elimination of stay plates and lacing on the diagonals reduces it. This increase in weight amounts to 2.5 per cent of the total structural steel; and, since the main objective was to conserve steel, the lighter design was adopted.

"The complete bracing system with the deck acting as a horizontal diaphragm and bracing in the vertical planes at the panel points and in the lower chord plane will give great torsional and lateral stiffness to the bridge.

"Section 3.6.67 of the AASHO Specification appears to require lateral bracing in the plane of the top chord, but Section 3.2.14 appears to allow for its omission. Because, the deck is exceptionally well keyed to the stringers and chords and adequate bracing is provided in a vertical plane at the panel points and in the bottom chord system, no top lateral braces were provided. The deck can carry the lateral loads at the top chord level with very low stresses resulting.

"The lateral bracing of the riveted bridge was governed by the maximum L/r of 240 for tension bracing members. If rods with clevises were used, the bracing would be lighter, but this would not be a riveted system. Rods and turnbuckles were used with pipe lateral struts (governed by L/r) for the welded bridge lower

lateral system.

"The lateral bracing in vertical planes (cross-frames) of the welded bridge will be explained in connection with the floorbeams.

"The welded bridge has weldments for rockers, shoes and bearing plates. At the fixed end the bearing plate permits deflection of the span without concentration of load at the edge of the bearing plate, and also is interlocked to take longitudinal and lateral loads. This design avoids all machining operations on the fixed end bearing plates except drilling the holes for the anchor bolts.

"The riveted bridge utilizes conventional cast steel rockers and pedestal shoes.

"The deck is concrete made with ordinary rock aggregate. A lighter deck with a consequent saving in steel could be made by using lightweight aggregate, but the larger shrinkage exhibited by lightweight concrete is undesirable from the standpoint of the composite action with steel members. Likewise, prestressing would save deck reinforcing steel, but would be costly and difficult to incorporate without introducing undesirable stresses in the stringers, chords, and floorbeams, and consequently was not used.

"The deck performs all of the following functions:

1. Running surface for traffic.
2. Stiffening diaphragm for lateral loads.
3. Top flange member of stringers and chords acting as composite beams.
4. Participating member with stringers and chords in carrying the compressive chord stress.

"The curb is reinforced and keyed to the deck so that it acts as an outside stringer or edge beam. Both the curb and the deck are reinforced longitudinally at the panel points to prevent cracking from negative moment. This reinforcement was considered in computing the negative moment resistance of the stringers.

"With the exception of the shear device at U_2, there are no hooked bars. The reinforcement mats can easily be laid out and tied, and there are only a few bar sizes and lengths.

"The stringers of both bridges were keyed to the deck so that the deck and stringers act together as composite beams. Dead load bending stresses were computed with the steel acting alone, because the dead load would be applied before the concrete has hardened. Live load bending stresses and direct compression stresses from action with the truss top chords were computed on the assumption

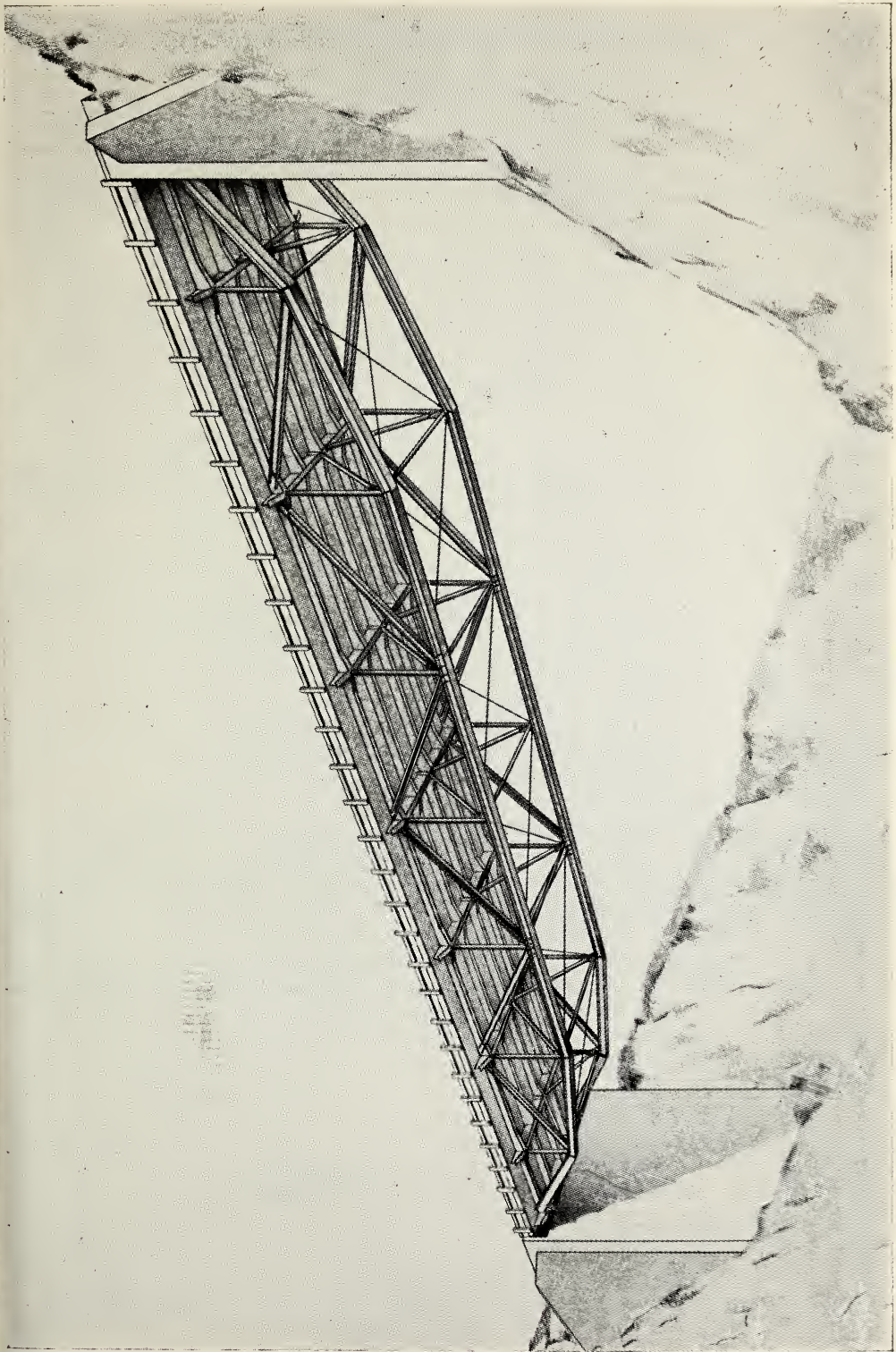
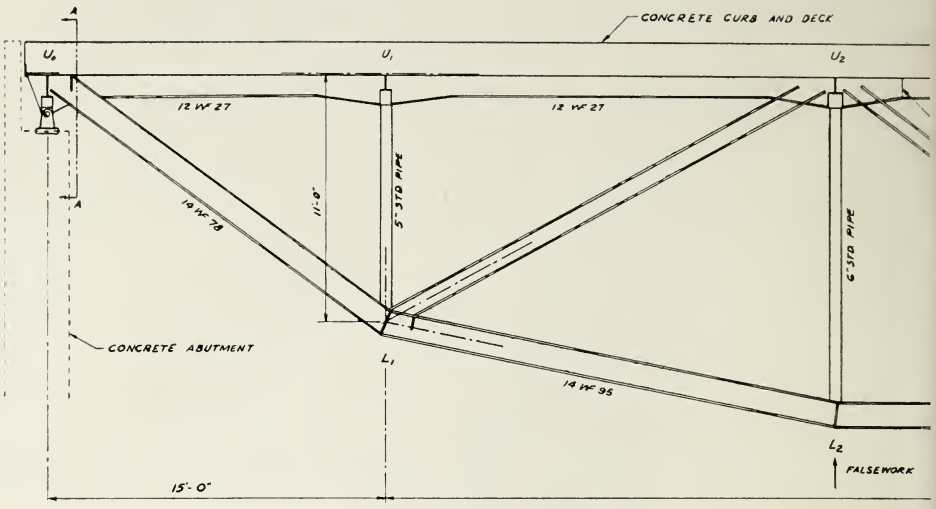
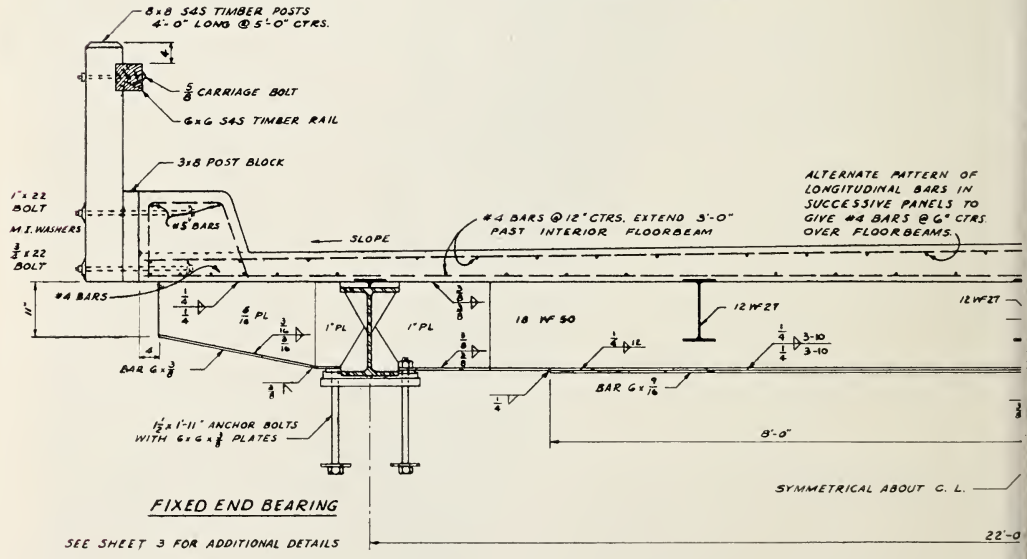


Figure 40



HALF EL.

TRUSSES TO BE SUPPORTED BY FALSEWORK AT L_2 AND L_3 OR AT EACH PANEL POINT DURING ERECTION AND POURING CONCRETE DECK AND CURBS. DECK FORMS TO BE SUPPORTED BY LOWER FLANGES OF STRINGERS AND FLOORBEAMS.


8x8 S4S TIMBER POSTS 4'-0" LONG @ 5'-0" CTRS.
5/8 CARRIAGE BOLT
6x6 S4S TIMBER RAIL
3x8 POST BLOCK
1"x 22 BOLT
M.I. WASHERS
3/4 x 22 BOLT
#5 BARS
SLOPE
#4 BARS @ 12" CTRS. EXTEND 3'-0" PAST INTERIOR FLOORBEAM
ALTERNATE PATTERN OF LONGITUDINAL BARS IN SUCCESSIVE PANELS TO GIVE #4 BARS @ 6" CTRS. OVER FLOORBEAMS.
#4 BARS
11"
6 PL 10
1" PL
1" PL
18 WF 50
12 WF 27
12 WF 27
BAR 6 x 3/8
BAR 6 x 9/16
1 1/2 x 1'-11" ANCHOR BOLTS WITH 6 x 6 x 3/8 PLATES
8'-0"
SYMMETRICAL ABOUT C. L.
22'-0

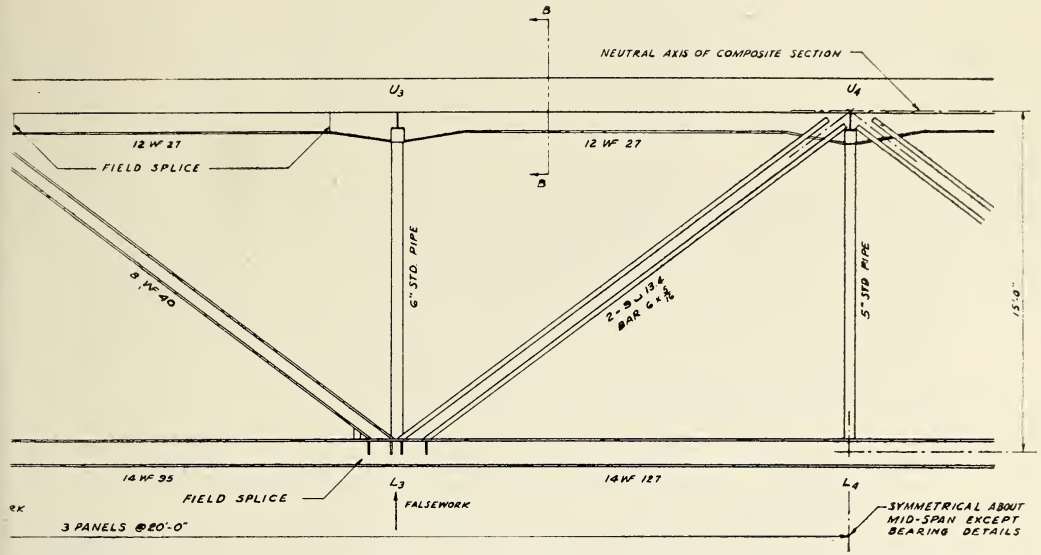
FIXED END BEARING

SEE SHEET 3 FOR ADDITIONAL DETAILS

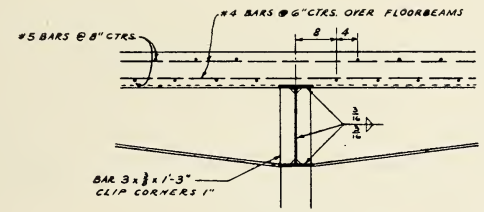
SECTION A-A
END FLOORBEAM

BASIC DESIGN STRESSES FOR CONCRETE:

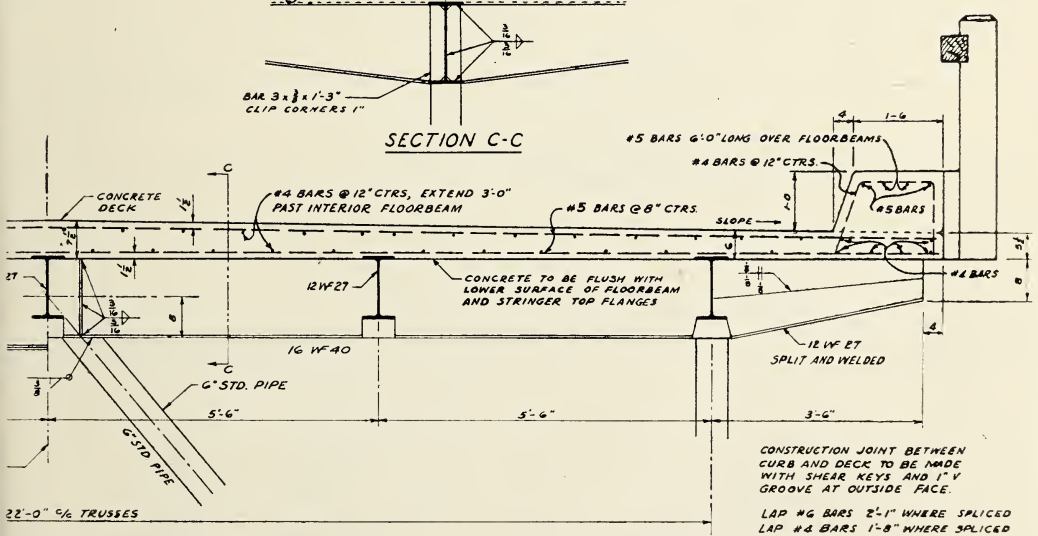
ALL CONCRETE TO TEST 3,000 psi. COMPRESSIVE STRENGTH AT 28 DAYS
f_c = 1200 psi. f_s = 20,000 psi. (INTERMEDIATE GRADE) N=10
BOND 150 psi., STRAIGHT BARS; 225 psi., HOOKED BARS.



ELEVATION

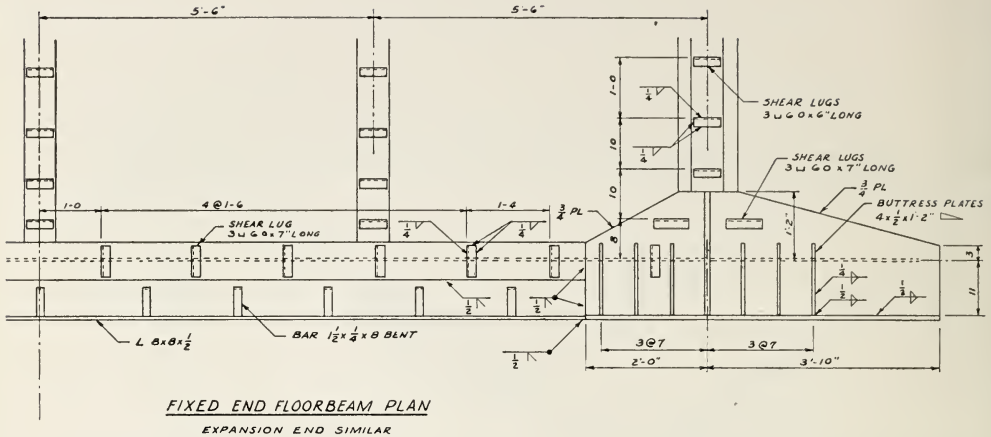


SECTION C-C


#5 BARS 6'-0" LONG OVER FLOORBEAMS
#4 BARS @ 12" CTRS.
#5 BARS
SLOPE
CONCRETE DECK
#4 BARS @ 12" CTRS, EXTEND 3'-0" PAST INTERIOR FLOORBEAM
#5 BARS @ 8" CTRS.
12WF27
CONCRETE TO BE FLUSH WITH LOWER SURFACE OF FLOORBEAM AND STRINGER TOP FLANGES
#4 BARS
16 WF 40
12 WF 27 SPLIT AND WELDED
6" STD. PIPE
5'-6"
5'-6"
3'-6"
6" STD. PIPE
22'-0" C/C TRUSSES
CONSTRUCTION JOINT BETWEEN CURB AND DECK TO BE MADE WITH SHEAR KEYS AND 1" V GROOVE AT OUTSIDE FACE.
LAP #6 BARS 2'-1" WHERE SPLICED
LAP #4 BARS 1'-8" WHERE SPLICED
SEE SHEET 5 FOR ADDITIONAL DETAILS.

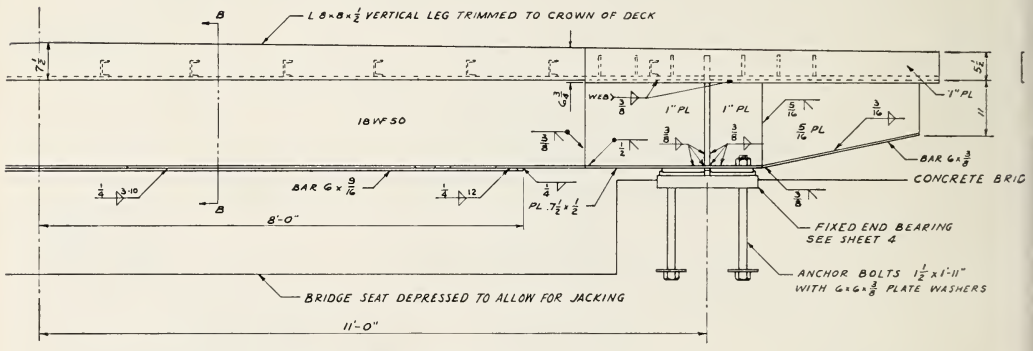
SECTION B-B
INTERIOR FLOORBEAM

Figure 41

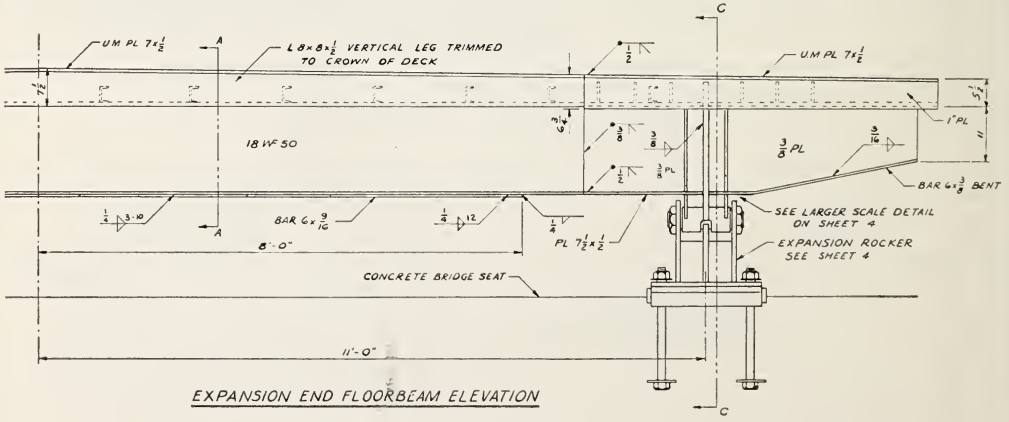

5'-6"
5'-6"
SHEAR LUGS 3 L G O x G" LONG
SHEAR LUGS 3 L G O x 7" LONG
BUTTRESS PLATES 4 x 1/2 x 1'-2"
4 @ 1-6
SHEAR LUG 3 L G O x 7" LONG
3/4 PL
L 8 x 8 x 1/2
BAR 12 1/2 x 1/4 x 8 BENT
3 @ 7
2'-0"
3'-10"

FIXED END FLOORBEAM PLAN

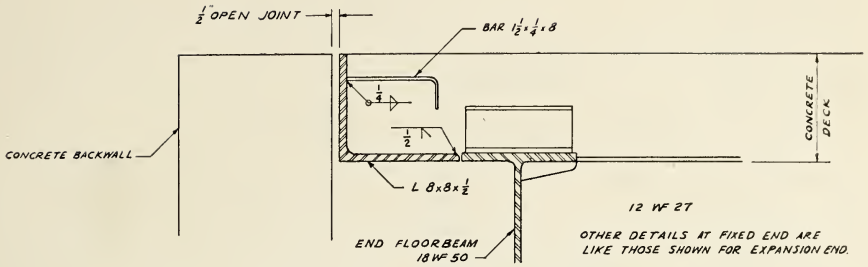
EXPANSION END SIMILAR


L 8 x 8 x 1/2 VERTICAL LEG TRIMMED TO CROWN OF DECK
18 WF 50
1" PL
BAR G x 3/8
CONCRETE BRIDGE SEAT
PL 7 1/2 x 1/2
FIXED END BEARING SEE SHEET 4
ANCHOR BOLTS 1 1/2 x 1'-11" WITH G x G x 3/8 PLATE WASHERS
BRIDGE SEAT DEPRESSED TO ALLOW FOR JACKING
8'-0"
11'-0"

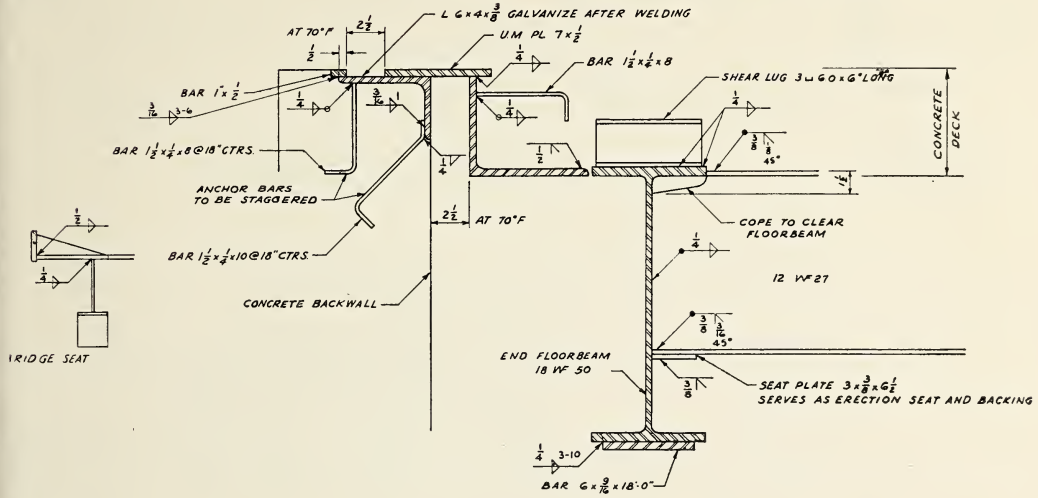
FIXED END FLOORBEAM ELEVATION


U.M. PL 7 x 1/2
L 8 x 8 x 1/2 VERTICAL LEG TRIMMED TO CROWN OF DECK
18 WF 50
1" PL
3/8 PL
BAR G x 3/8 BENT
BAR G x 9/16
PL 7 1/2 x 1/2
SEE LARGER SCALE DETAIL ON SHEET 4
EXPANSION ROCKER SEE SHEET 4
CONCRETE BRIDGE SEAT
8'-0"
11'-0"

EXPANSION END FLOORBEAM ELEVATION

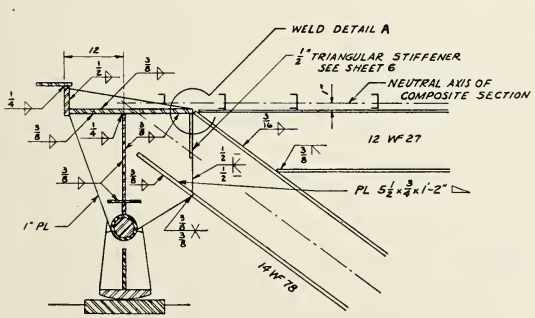

½" OPEN JOINT
BAR 1½ x ¼ x 8
CONCRETE BACKWALL
CONCRETE DECK
L 8 x 8 x ½
12 WF 27
END FLOORBEAM 18 WF 50
OTHER DETAILS AT FIXED END ARE LIKE THOSE SHOWN FOR EXPANSION END.

FIXED END JOINT DETAIL

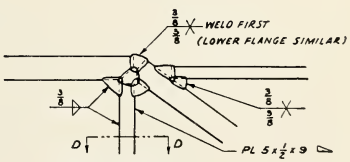

L 6 x 4 x ⅜ GALVANIZE AFTER WELDING
AT 70°F
2½
UM PL 7 x ½
BAR 1 x ½
BAR 1½ x ¼ x 8
SHEAR LUG 3 in. G x G" LONG
CONCRETE DECK
BAR 1½ x ¼ x 8 @ 18" CTRS.
ANCHOR BARS TO BE STAGGERED
AT 70°F
COPE TO CLEAR FLOORBEAM
12 WF 27
BAR 1½ x ⅜ x 10 @ 18" CTRS.
CONCRETE BACKWALL
IRIDGE SEAT
END FLOORBEAM 18 WF 50
SEAT PLATE 3 x ⅜ x G½ SERVES AS ERECTION SEAT AND BACKING
BAR G x 9/16 x 18'-0"

SECTION A-A

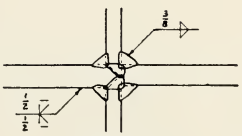
EXPANSION JOINT DETAIL AND INTERIOR STRINGER CONNECTION TO END FLOORBEAM



EXPANSION END BEARING SEC. C-C

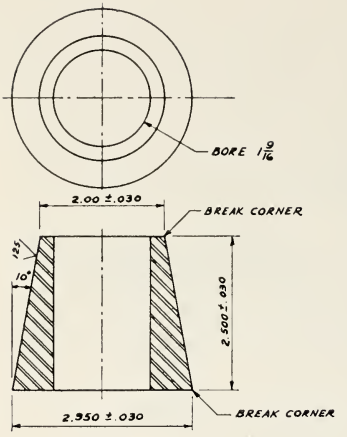


WELD DETAIL A



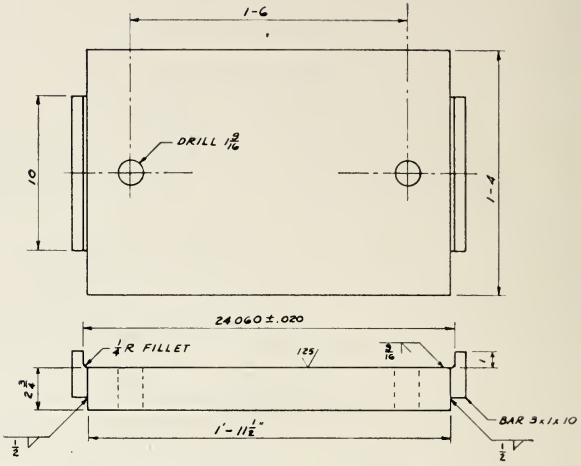
SEC D-D

Figure 42



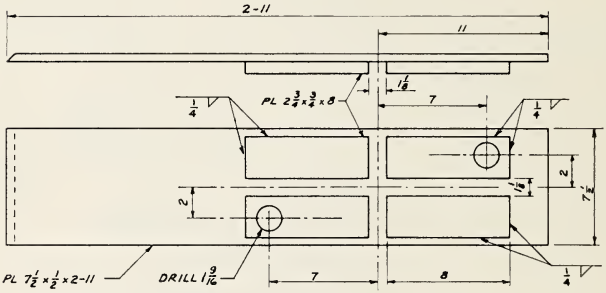
CONE SPACER

4 REQUIRED
WEIGHT 2.2 LB EACH



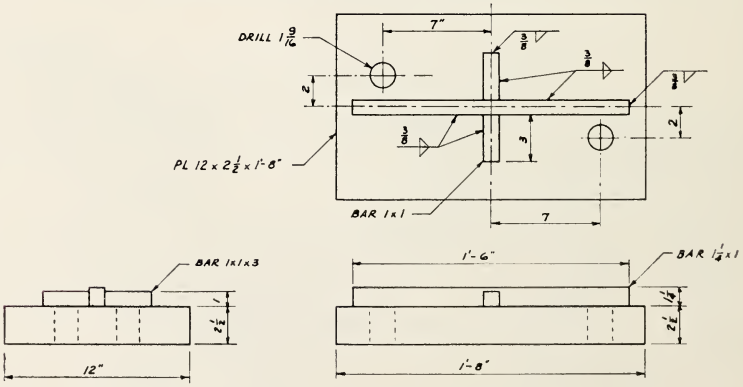
EXPANSION END BEARING PLATE

2 REQUIRED - WEIGHT 307 LB EACH



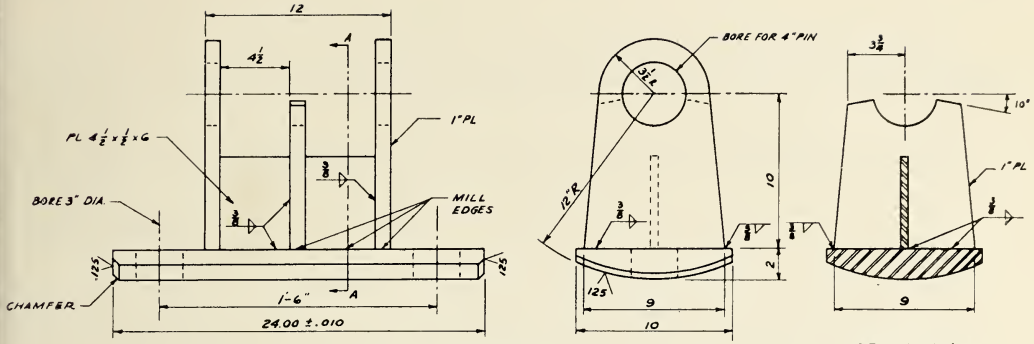
FIXED END UPPER BEARING PLATE

2 REQUIRED - RIGHT AND LEFT HAND



FIXED END LOWER BEARING PLATE

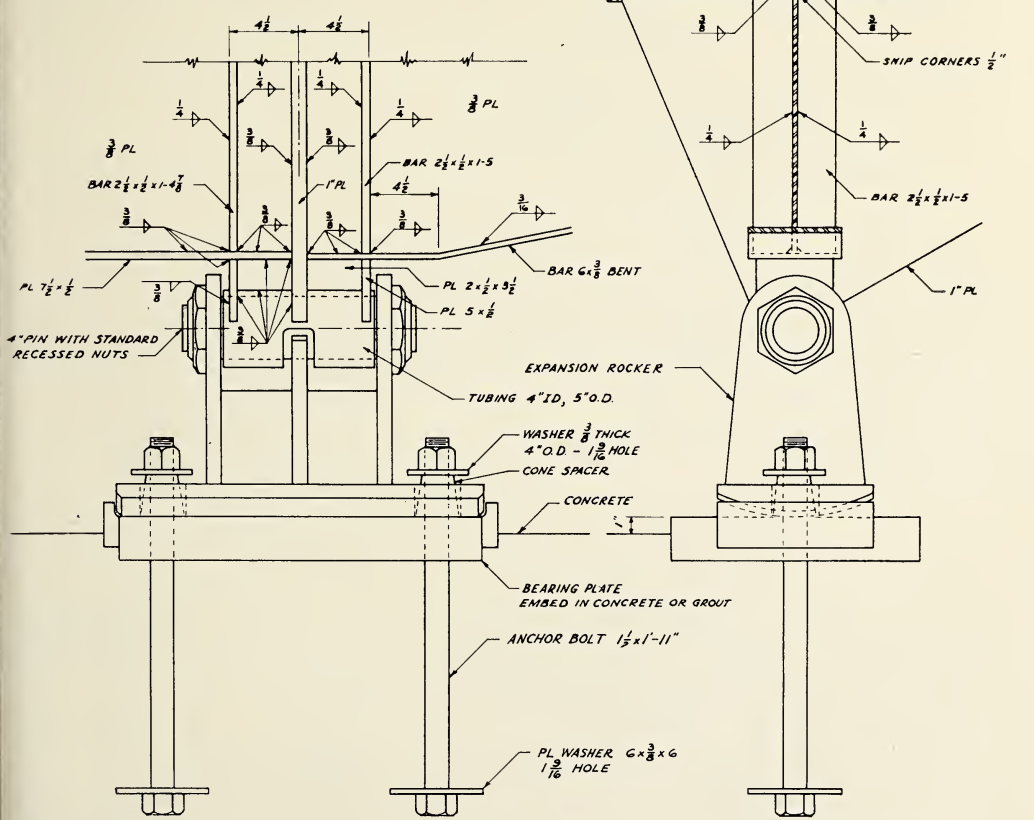
2 REQUIRED - RIGHT AND LEFT HAND
WEIGHT 1764 LB. EACH



EXPANSION ROCKER

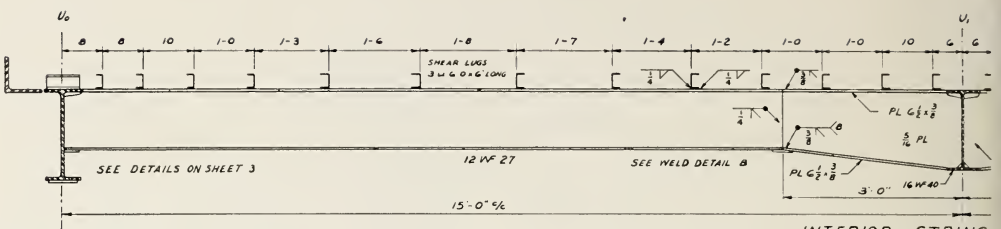
2 REQUIRED - WEIGHT 187.4 LB EACH

SECTION A-A

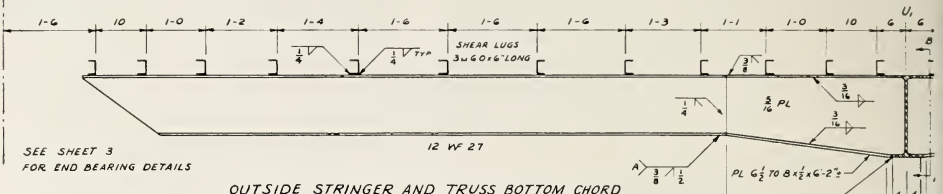


EXPANSION END BEARING DETAILS

Figure 43



INTERIOR STRING

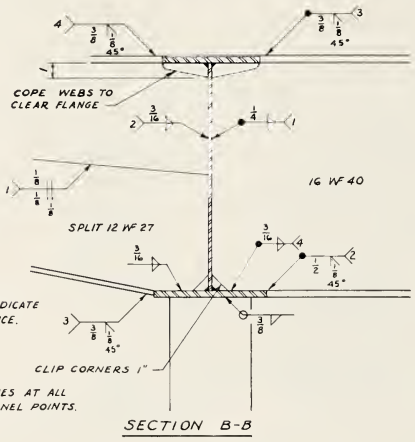


OUTSIDE STRINGER AND TRUSS BOTTOM CHORD

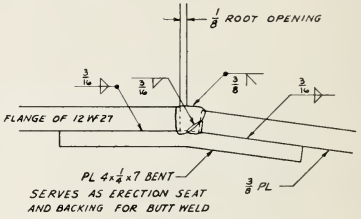
A. SEE FULL SIZE WELD DETAIL A, SHEET G.

5" STD PIPE

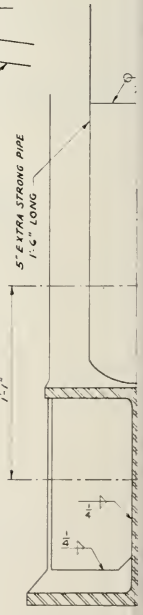
JOINT U

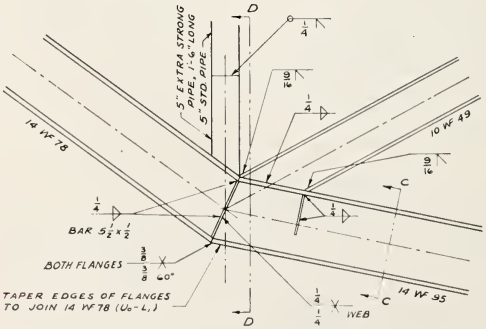

COPE WEBS TO CLEAR FLANGE
16 WF 40
SPLIT 12 WF 27
NOTE: NUMBERS INDICATE WELDING SEQUENCE. (TYPICAL)
CLIP CORNERS 1"
THIS DETAIL APPLIES AT ALL OTHER INTERIOR PANEL POINTS.

SECTION B-B

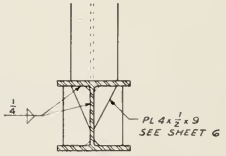


WELD DETAIL B



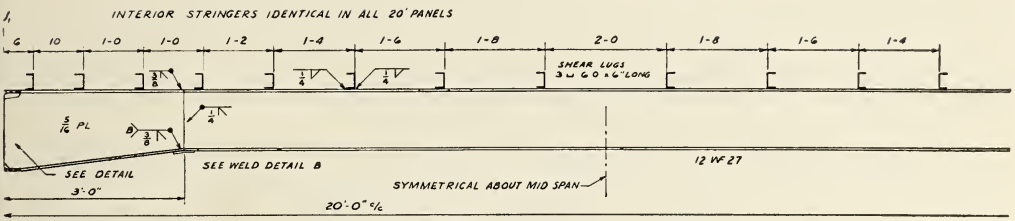

5" EXTRA STRONG PIPE, 1'-6" LONG
5" STD PIPE
14 WF 78
10 WF 49
BAR 5½ x ½
BOTH FLANGES
TAPER EDGES OF FLANGES TO JOIN 14 WF 78 (U6 - L1)
14 WF 95
WEB

JOINT L1

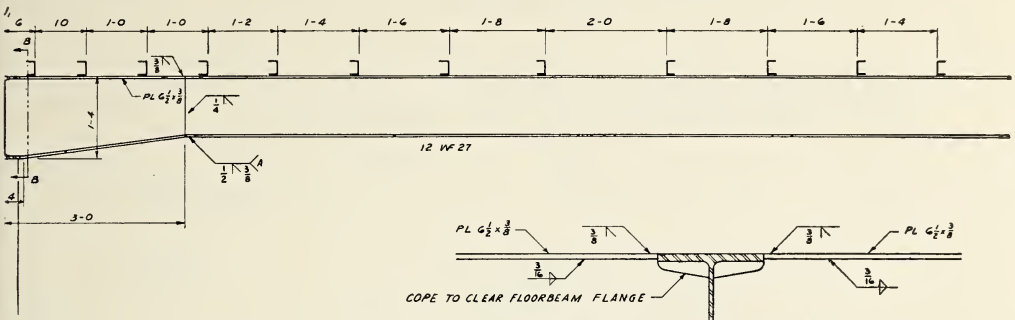


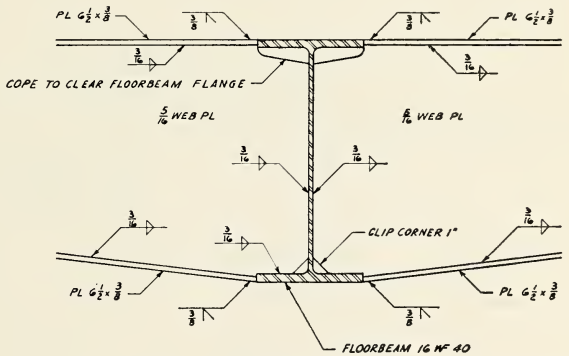
SECTION C-C

SECTI

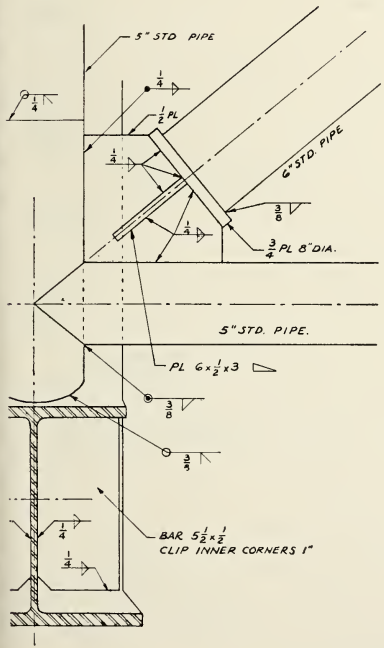

INTERIOR STRINGERS IDENTICAL IN ALL 20' PANELS
SHEAR LUGS 3 ⌴ G 0 x 6" LONG
SEE DETAIL
3'-0"
SEE WELD DETAIL B
12 WF 27
SYMMETRICAL ABOUT MID SPAN
20'-0" ℄

NGERS (3 LINES)

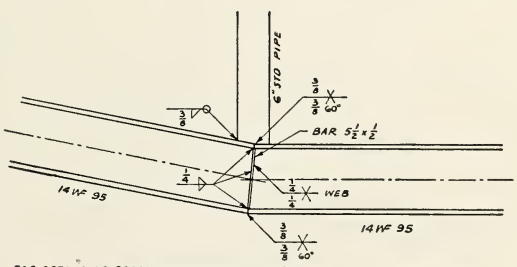




TYPICAL CONNECTION OF STRINGERS TO FLOORBEAM



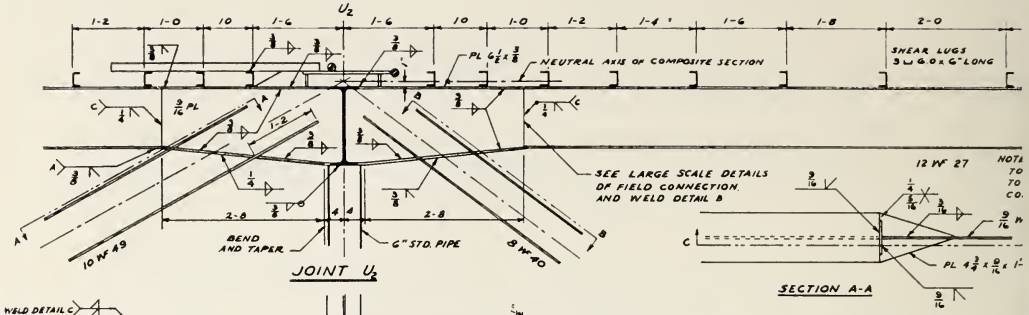
TION D-D



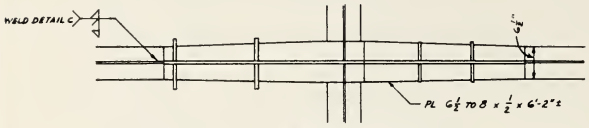
FOR DETAILS OF CROSS FRAME CONNECTION SEE JOINT L_4, TRANSVERSE SECTION SHEET 7

JOINT L_2

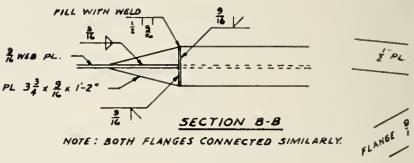
Figure 44

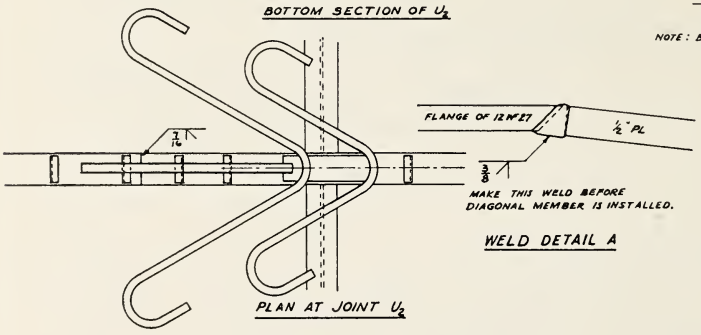

U2
SHEAR LUGS 3 L 6.0 x 6"-LONG
NEUTRAL AXIS OF COMPOSITE SECTION
PL 6½ x ⅜
SEE LARGE SCALE DETAILS OF FIELD CONNECTION AND WELD DETAIL B
BEND AND TAPER
6" STD. PIPE
10 WF 49
8 WF 40
12 WF 27

JOINT U_2



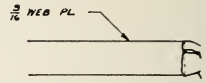
BOTTOM SECTION OF U_2

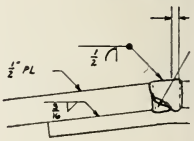


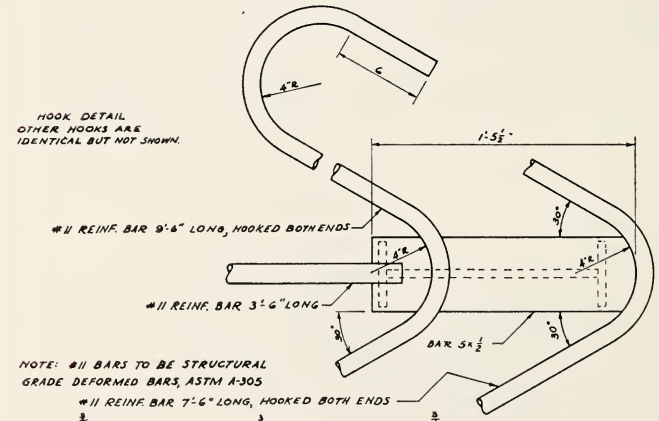


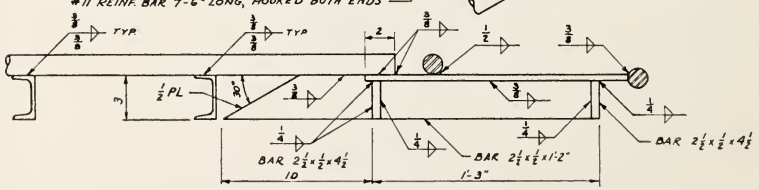
WELD DETAIL A

PLAN AT JOINT U_2

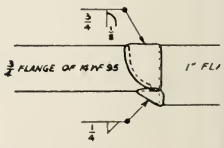


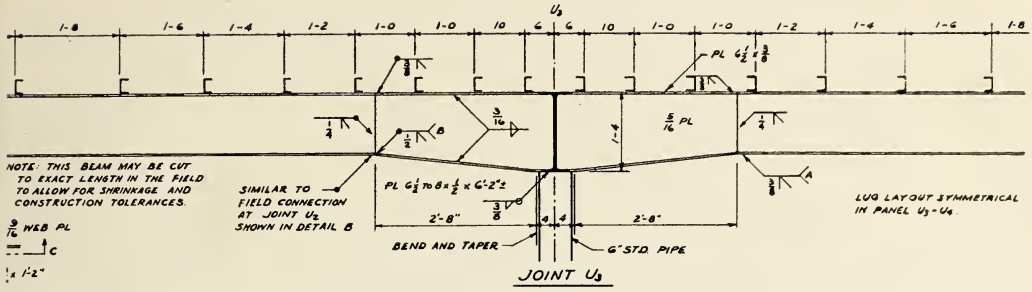



HOOK DETAIL
OTHER HOOKS ARE IDENTICAL BUT NOT SHOWN.
#11 REINF. BAR 9'-6" LONG, HOOKED BOTH ENDS
#11 REINF. BAR 3'-6" LONG
BAR 5 x ½
NOTE: #11 BARS TO BE STRUCTURAL GRADE DEFORMED BARS, ASTM A-305
#11 REINF. BAR 7'-6" LONG, HOOKED BOTH ENDS

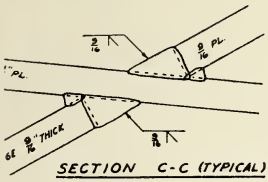


JOINT U_2 SHEAR DEVICE

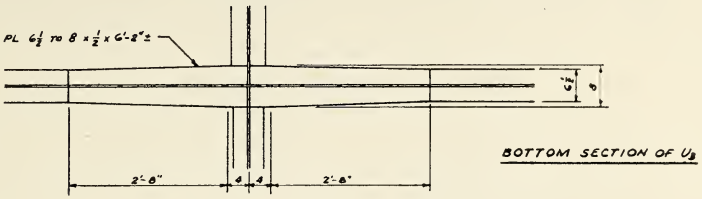




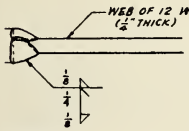
JOINT U_3



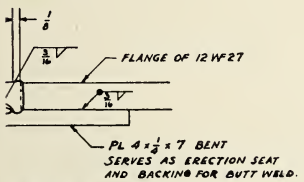
SECTION C-C (TYPICAL)



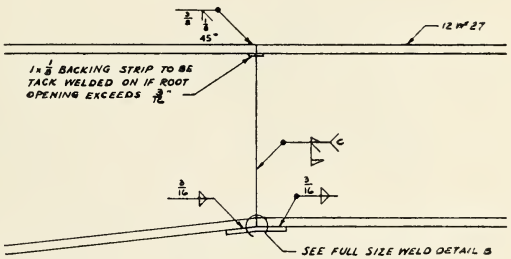
BOTTOM SECTION OF U_3



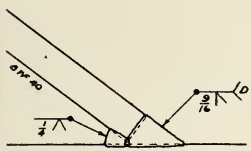
TAIL C

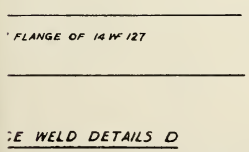


DETAIL B

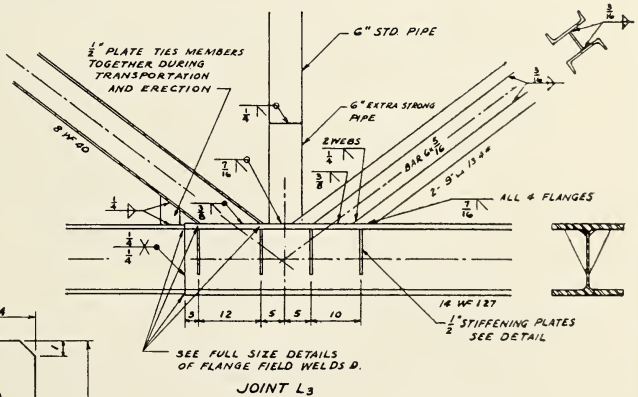


FIELD CONNECTION DETAILS

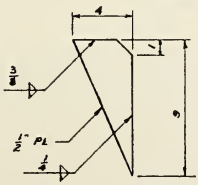




E WELD DETAILS D

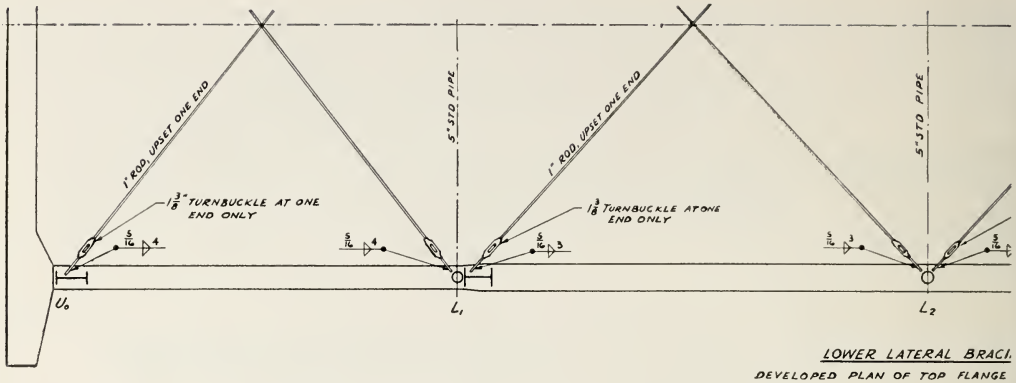


JOINT L_3



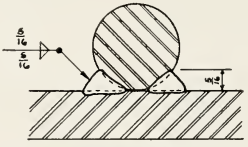
STIFFENING PLATE

Figure 45



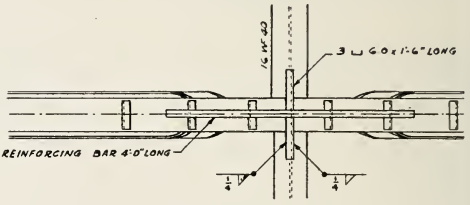
LOWER LATERAL BRACI

DEVELOPED PLAN OF TOP FLANGE

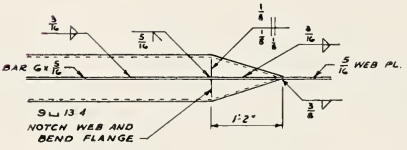


FULL SIZE SECTION

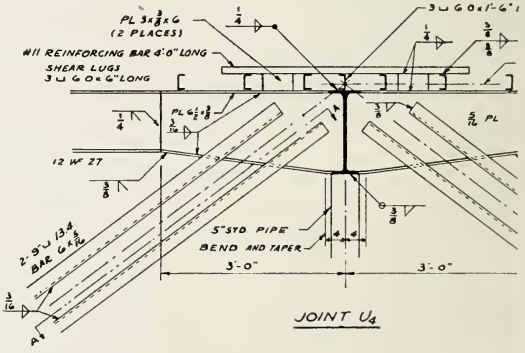
WELDS CONNECTING LOWER LATERAL RODS TO LOWER CHORD TOP FLANGE



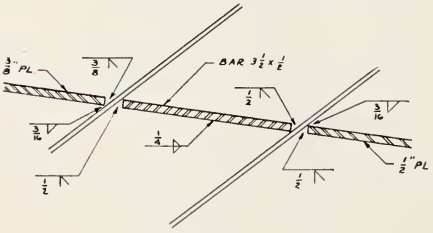
PLAN - JOINT U_4



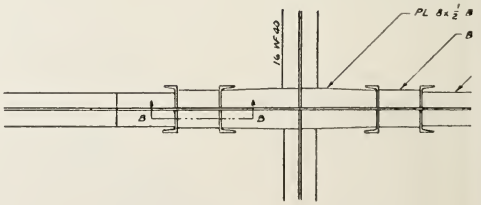
SECTION A-A


PL 3 3/8 x 6 (2 PLACES)
#11 REINFORCING BAR 4'-0" LONG
SHEAR LUGS 3 ⌴ 6 0 x 6" LONG
3 ⌴ 6 0 x 1'-6" L
PL 6 1/2 x 1/8
5/16 PL
12 WF 27
5" STD PIPE BEND AND TAPER
2 - 9 ⌴ 13.4 BAR 6 x 5/16
3'-0"
3'-0"
A

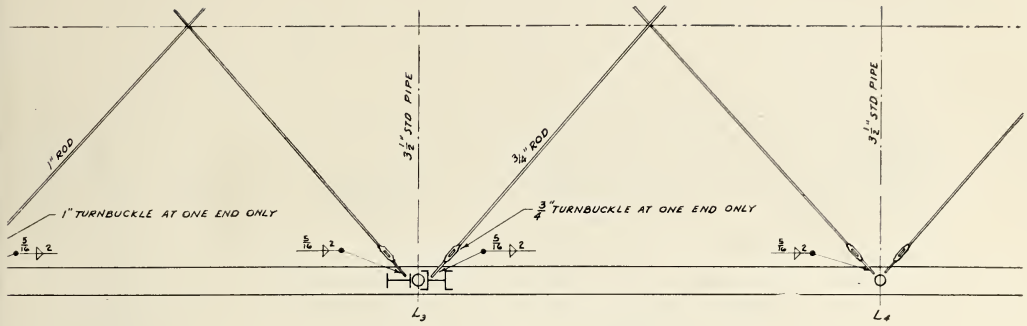
JOINT U_4



SECTION B-B



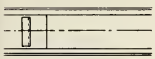
BOTTOM SECTION - JOINT U_4

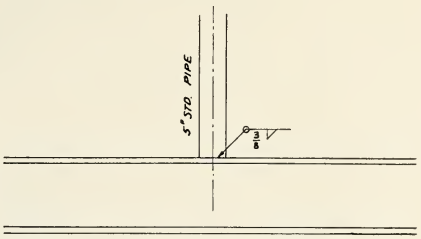


RACING SYSTEM

NGE OF LOWER CHORD

ONG





JOINT L_4

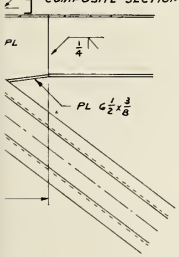
1'-6" LONG

TYP.

NEUTRAL AXIS OF COMPOSITE SECTION

PL

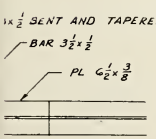
PL 6½x⅜

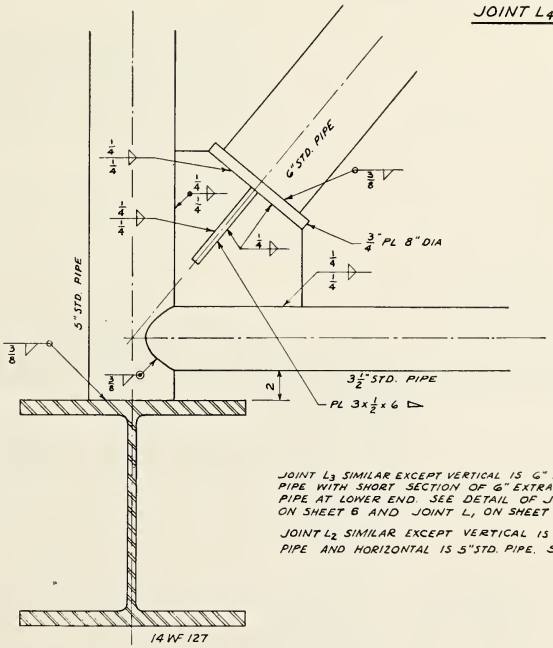


1x½ SENT AND TAPERED TO 7¼ WIDE

BAR 3½x½

PL 6½x⅜





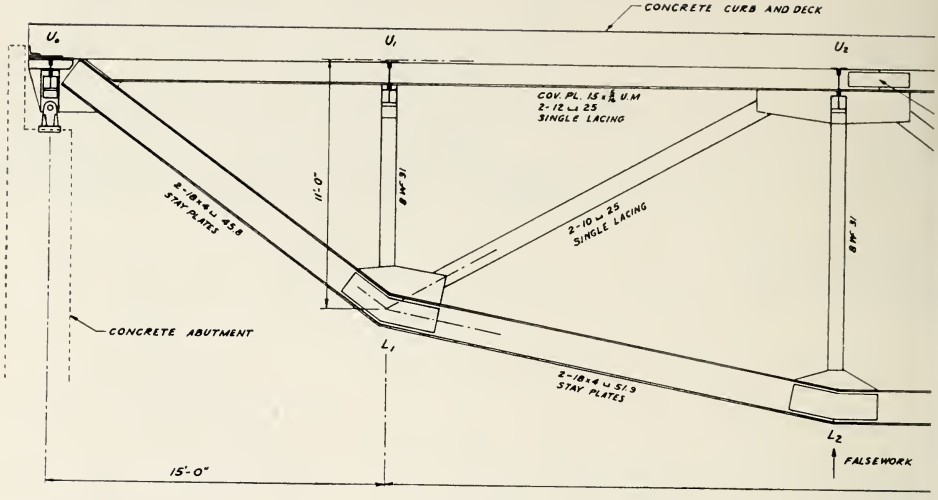
JOINT L_3 SIMILAR EXCEPT VERTICAL IS 6" STD. PIPE WITH SHORT SECTION OF 6" EXTRA STRONG PIPE AT LOWER END. SEE DETAIL OF JOINT L_3 ON SHEET 6 AND JOINT L_1 ON SHEET 5.

JOINT L_2 SIMILAR EXCEPT VERTICAL IS 6" STD. PIPE AND HORIZONTAL IS 5" STD. PIPE. SEE SHEET 5.

TRANSVERSE SECTION

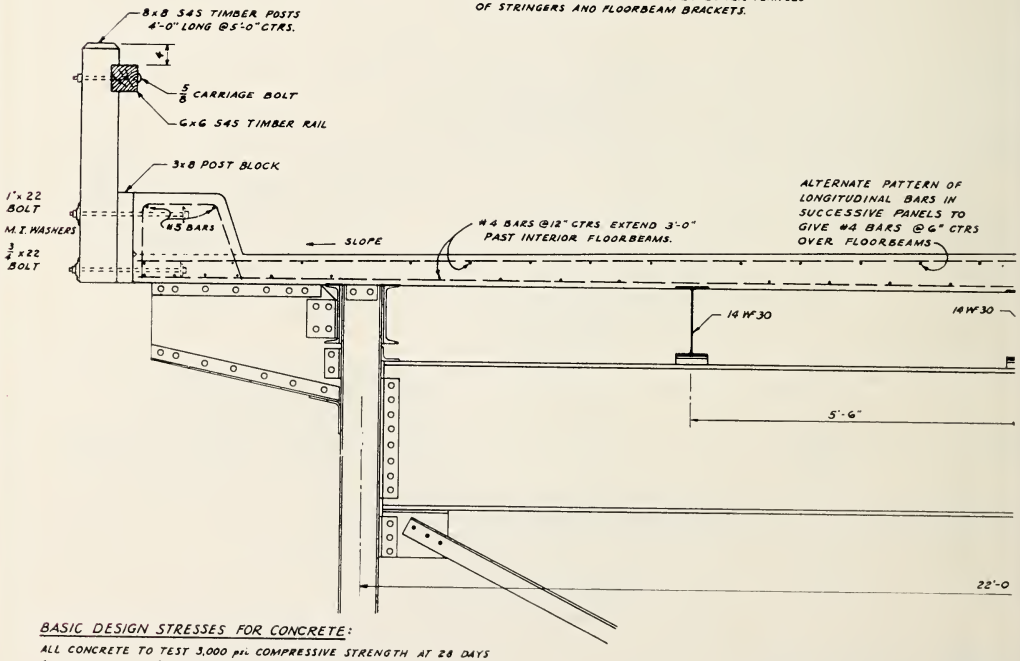
JOINT L_4

Figure 46



HALF EL.

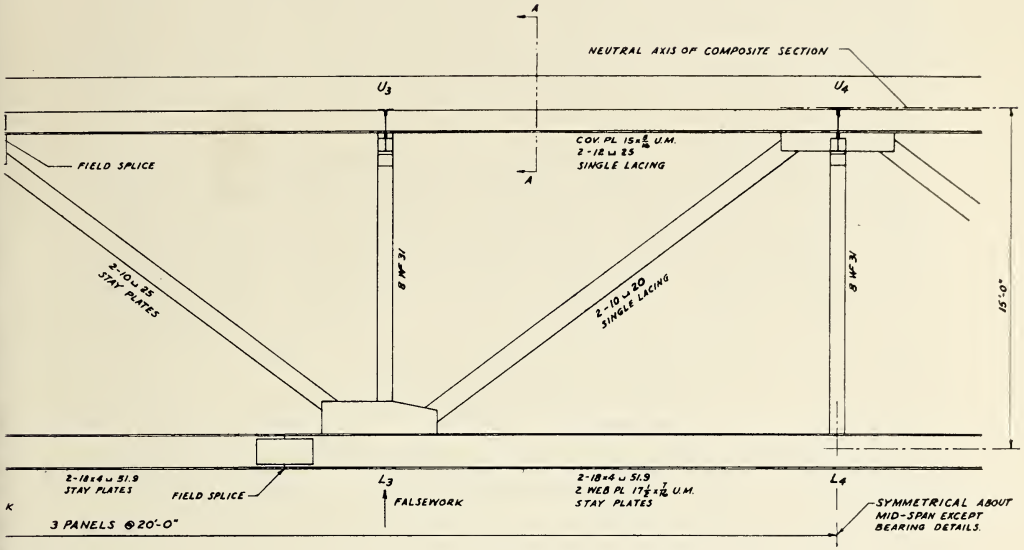
TRUSSES TO BE SUPPORTED BY FALSEWORK AT L_2 AND L_3, OR AT EACH PANEL POINT, DURING ERECTION OF STEEL AND POURING OF CONCRETE DECK AND CURBS.
DECK FORMS TO BE SUPPORTED BY LOWER FLANGES OF STRINGERS AND FLOORBEAM BRACKETS.


8x8 S4S TIMBER POSTS
4'-0" LONG @ 5'-0" CTRS.
5/8 CARRIAGE BOLT
6x6 S4S TIMBER RAIL
3x8 POST BLOCK
1" x 22 BOLT
M.I. WASHERS
3/4 x 22 BOLT
#3 BARS
SLOPE
#4 BARS @ 12" CTRS EXTEND 3'-0" PAST INTERIOR FLOORBEAMS.
ALTERNATE PATTERN OF LONGITUDINAL BARS IN SUCCESSIVE PANELS TO GIVE #4 BARS @ 6" CTRS OVER FLOORBEAMS
14 WF 30
14 WF 30
5'-6"
22'-0

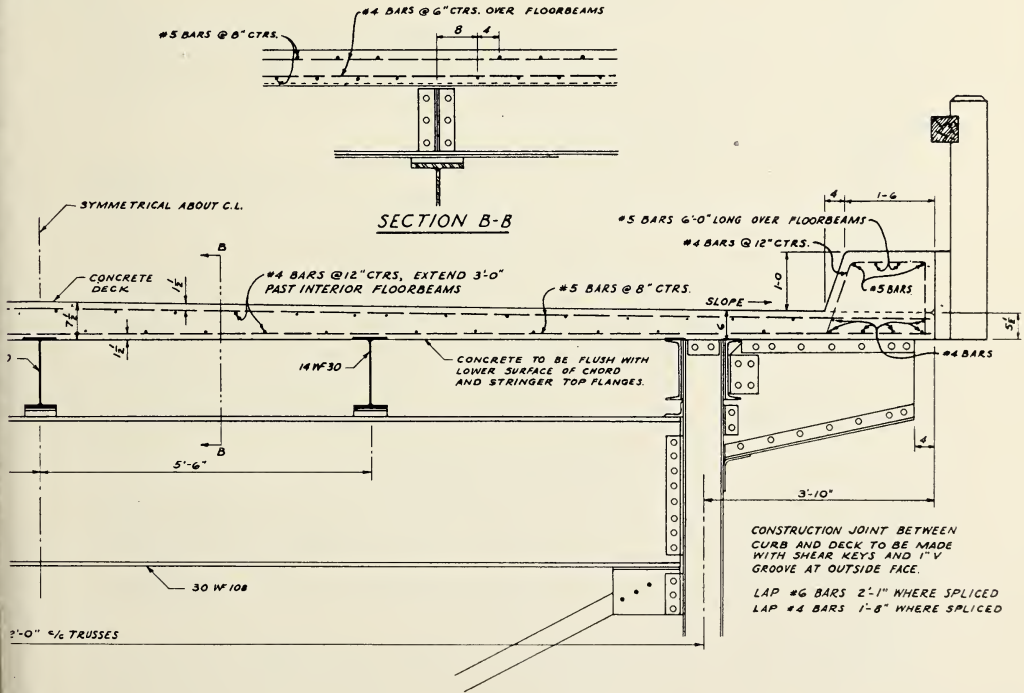
BASIC DESIGN STRESSES FOR CONCRETE:

ALL CONCRETE TO TEST 3,000 psi. COMPRESSIVE STRENGTH AT 28 DAYS
f_c = 1000 psi. f_s = 20,000 psi. (INTERMEDIATE GRADE) N = 10
BOND: 150 psi., STRAIGHT BARS; 225 psi., HOOKED BARS

SECTI



ELEVATION


#4 BARS @ 6" CTRS. OVER FLOORBEAMS
#5 BARS @ 8" CTRS.
8
4
SECTION B-B
SYMMETRICAL ABOUT C.L.
#5 BARS 6'-0" LONG OVER FLOORBEAMS
#4 BARS @ 12" CTRS.
#5 BARS
CONCRETE DECK
#4 BARS @12"CTRS, EXTEND 3'-0" PAST INTERIOR FLOORBEAMS
#5 BARS @ 8" CTRS.
SLOPE
#4 BARS
14 WF 30
CONCRETE TO BE FLUSH WITH LOWER SURFACE OF CHORD AND STRINGER TOP FLANGES.
5'-6"
3'-10"
CONSTRUCTION JOINT BETWEEN CURB AND DECK TO BE MADE WITH SHEAR KEYS AND 1" V GROOVE AT OUTSIDE FACE.
LAP #6 BARS 2'-1" WHERE SPLICED
LAP #4 BARS 1'-6" WHERE SPLICED
30 WF 108
'-0" c/c TRUSSES

TION A-A

Figure 47

that no sliding takes place between deck and stringers. Horizontal shear loads applied at one panel point were considered to be uniformly distributed over the full width of the deck at the adjacent panel point and beyond. The top flanges of the stringers were designed to have lateral support from the concrete deck by making the bottom surface of the deck flush with the lower faces of the stringer top flanges.

"The stringers of the welded bridge were deepened at the interior floorbeams to carry negative moment and to make them the same depth as the floorbeams, thus giving lateral support to the lower flanges of the floorbeams. The welding specifications limit the tension stress to 75 per cent of the basic value (13,500 psi) at points where there are abrupt changes in section or transverse attachments. However, the governing stress in the stringers at panel points was the 18,000 psi allowable compression resulting from combined and direct chord compression.

"The stringers of the riveted bridge were placed over the tops of the floorbeams and were spliced near points of contraflexure. This should give a close approach to the integral action with the deck achieved in the welded design, and would be very easy to erect. Another reason for placing the stringers on top of the floorbeams was that this located the floorbeams below the top chord channels and avoided congestion at the joints. Obviously the floorbeams could not be on top of the chords if the chords were to act with the deck as a composite member.

"The combined bending and compression load at panel points requires a cover plate on the bottom flange of the stringers of the riveted bridge at the support, and web stiffeners were provided at supports to prevent local buckling of the webs.

"The interior floorbeams of the welded bridge are supported at mid-span by diagonal struts which carry load to the lower chord panel points. Pipes were used for the struts to obtain maximum radius of gyration. This arrangement saves considerable steel in the floorbeams by making them continuous beams with 11 ft. spans instead of the 22 ft. span between trusses. Moreover, the verticals at panel points 1, 2, and 3 were made lighter by this arrangement which divides the load between upper and lower panel points. The struts also function as a part of the bracing system and thereby achieve the economy of material that attends the use of a single member for more than one function.

"Although the author attempted to design the riveted floorbeams in the same way as the welded ones, this proved to be impractical.

The diagonal struts could not be connected rigidly to the floorbeams by any simple connection and with the stringers above the floorbeams the lower flanges of the floorbeams lacked lateral support. This would be an unsafe condition in a region of high compression from negative moment. Consequently the interior floorbeams of the riveted bridge were designed as simple beams with a span of 22 ft."

Figures 41 to 46, inclusive show the details of the welded bridge. Figure 47 shows an elevation and cross section of the riveted bridge.

Mr. Jennison gave elaborate cost studies, detailed lists of material, and even made comparisons between the use of A 7 steel and A 242 low alloy steel. Summaries for the riveted and welded bridges are as follows:

RIVETED BRIDGE

Structural Steel, Shapes and Plate			
Structural Shapes			124,151 lb.
Plate			32,179 lb.
Bearing Plates			790 lb.
		Subtotal	157,120 lb.
Rockers and Pedestal Shoes			1,568 lb.
Miscellaneous Iron and Steel (Pins, pin nuts, bolts, washers, rivet heads)			5,198 lb.
		Subtotal	163,886 lb.
Reinforcing Steel			21,461 lb.
Total Structural and Reinforcing Steel			185,347 lb.
Rivets	*Shop*	*Field*	*Total*
7/8"	10,186	1,624	11,810
3/4"	1,312	....	1,312
Concrete Deck Slab			108 cu. yd.
Timber, Railing			2,237 fbm

WELDED BRIDGE

Structural Steel, Shapes and Plate		
Structural Shapes		70,498 lb.
Plate		10,820 lb.
Bearing Plates and Rockers		1,342 lb.
	Subtotal	82,660 lb.

Pipe, Structural Members	8,732 lb.
Subtotal	91,392 lb.
Miscellaneous Iron and Steel (Pins, pin nuts, bolts, washers, turnbuckles)	1,000 lb.
Reinforcing Steel	21,578 lb.
Total Structural and Reinforcing Steel	113,970 lb.
Welding Rod 781 lb. (shop) + 151 lb. (field) =	932 lb.
Concrete, Deck Slab	108 cu. yd.
Timber, Railings	2,237 fbm

Adding up all of the contributing items, the estimated cost of the erected structural steel is $0.21 per lb. for the riveted structure and $0.247 per lb. for the welded structure.

R. W. Ullman, Cleveland Heights, Ohio, selected a 26 ft. roadway and a span of 130 ft. for his bridge. Also, he choose to design the reinforced concrete slab to act compositely with the longitudinal deck trusses. The slab is 9 in. thick at the bridge centerline, but because of a 2 in. crown, is 7 in. thick at the curbs. It spans the 9 ft. between the four lines of truss top chords without the need of stringers or floorbeams.

The riveted bridge utilizes four vertical trusses spaced at 9 ft. centers as shown in Figure 48. The welded bridge consists of six inclined trusses that are arranged to form three triangular space frames as shown in Figure 49. The depth of both types of trusses is 7 ft.-9½ in.

Mr. Ullman described his welded structure as follows:

"The bridge consists of a concrete roadway slab securely attached to three triangular units 'U' (see Figure 49). Each of these three units is comprised of two inclined trusses of the Warren type which are cross-connected at the top, every 8 ft.-1½ in. These Units 'U' are to be erected as one unit, then attached to each other by completing the welding of the shear lugs across the top flange of the adjacent Unit. The bottom laterals, space at 16 ft.-3 in. center to center, are then field welded in and the supporting deck structure is complete. From the typical cross section, it will be seen that the supporting deck structure is broken down into five isosceles triangles in which the inclined trusses are the load carrying members of the bridge, and the concrete deck is the compressive member of the entire span.

"This unique arrangement results in many advantages over

present methods of design:

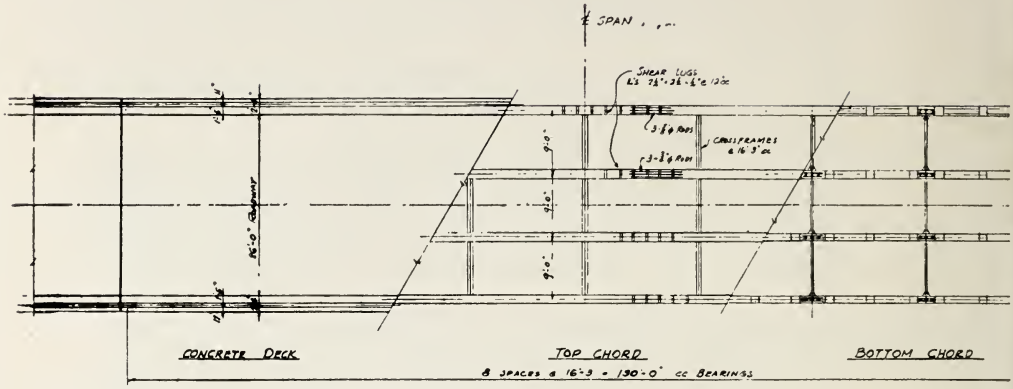
1. Great strength and rigidity because of the triangular breakdown of the supporting system in longitudinal and transverse directions.
2. No cross frames are required because the inclined members, leaning against each other provide at the same time lateral stability and a very effective load transfer.
3. All interior loads are always taken by two inclined members against one in the vertical design. Thus a load P, for which a vertical member must be designed, reduces in the inclined design to $\frac{P}{2 \sin \alpha}$ in which the angle α is the angle between the inclined member and the horizontal.
4. An overall economy, in comparison with an identical design of vertical truss riveted construction, which is surprising. This is attributable to the advantages quoted in paragraphs '2' and '3', the elimination of punching and precision shop layout necessary in riveted construction and other advantages gained by the use of the arc welding process.

* * *

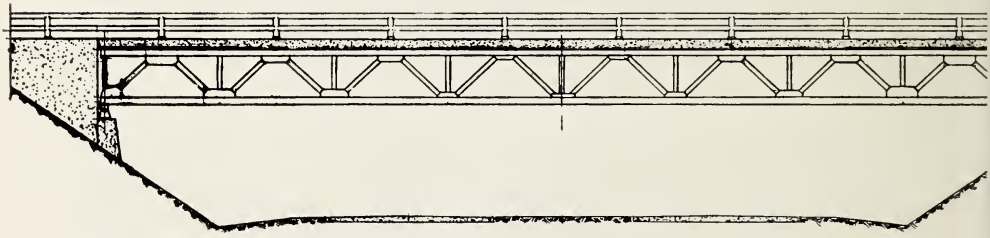
"Diagonals and posts are all wide flange T sections welded to the inclined sides of the top and bottom chords. The top and bottom chords consist of 14 x ½ in. plates and 8 in. plates of varying thicknesses welded to each other at an angle of 60°. Because of composite action, the horizontal top chord plates vary in thickness only from ⅜ in. to ½ in., while the bottom chord horizontal plates vary from ⅜ in. to 1¾ in.

"To the top chord, 2½ x ½ in. shear lugs are welded at 12 in. center to center. To these shear lugs, three longitudinal deformed bars are welded. This device will attach the concrete slab securely to the steel work so that a composite action is assured.

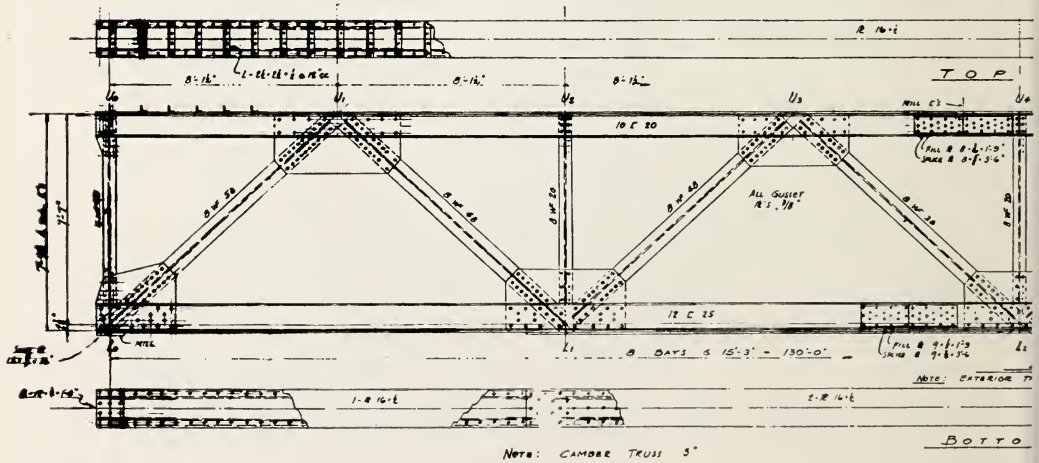
"As shown in Figure 50 the top and bottom chord plates are first fabricated to a length of 131 feet by butt welding. The 14 x ½ in. and 8 in. wide plates are then machine welded to each other on an angle of 60°. To the long leg of the angle thus formed, the diagonals and posts are fillet welded to the face of the 14 x ½ plate and later, after turning the truss over, across the back of the T section. The entire deck structure is, practically, made up of six identical trusses which are easy to fabricate, with most of the welding done in flat position (see Typical Truss Details, Figure 50).



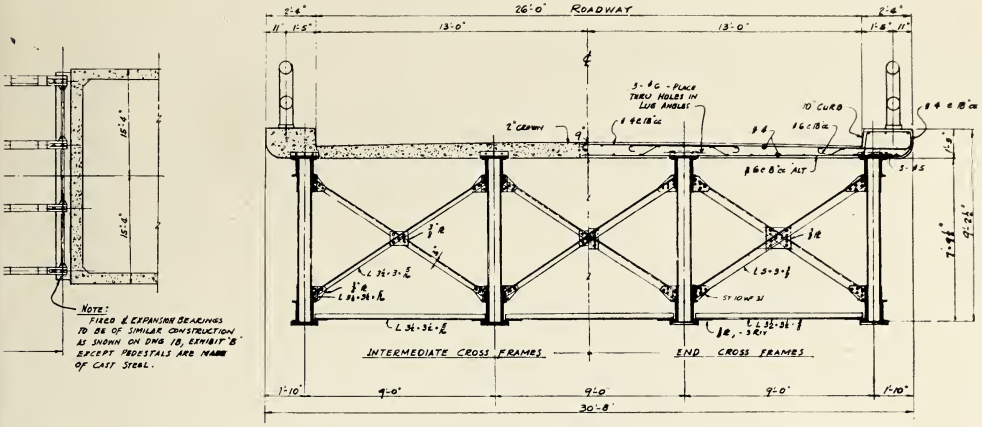
PLAN



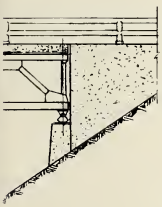
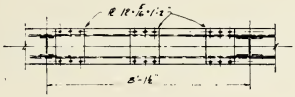
ELEVATION



NOTE: CAMBER TRUSS 5"


26'-0" ROADWAY
2'-6"
11'-5"
13'-0"
13'-0"
1'-8"
2" CROWN
10" CURB
3 - 1" ø PLACE THRU HOLES IN LUG ANGLE
INTERMEDIATE CROSS FRAMES
END CROSS FRAMES
30'-8"
NOTE: FIXED & EXPANSION BEARINGS TO BE OF SIMILAR CONSTRUCTION AS SHOWN ON DWG 18, EXHIBIT 'B' EXCEPT PEDESTALS ARE NAME OF CAST STEEL.

TYPICAL CROSS SECTION



TYPICAL - BOTTOM OF TOP CHORD
Sc. 1/2"-1'-0"

TYPICAL - TOP OF BOTTOM CHORD

See Design, Pages 16, 17, 18

NOTES:

DESIGN IN ACCORDANCE WITH STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, [illegible] EDITION.

L/a = < 140 FOR BRACING MEMBERS
L/a = < 120 " ALL OTHER "

UNIT STRESSES USED:

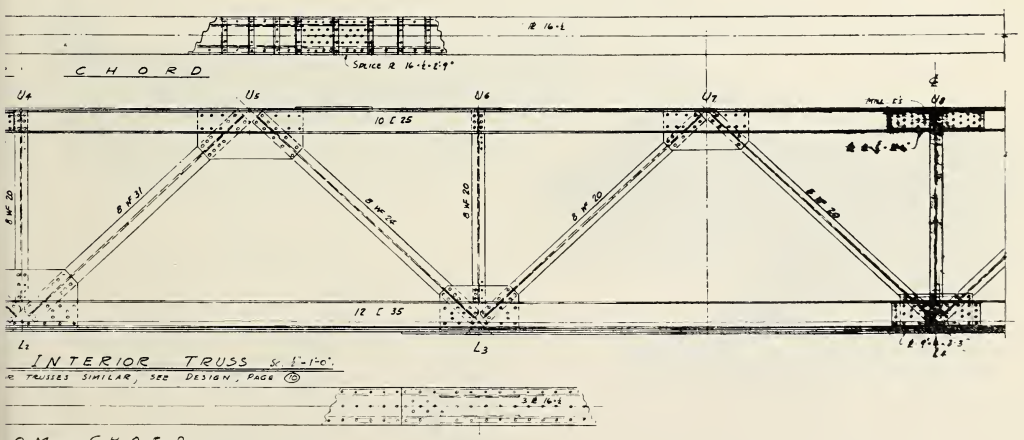
CONCRETE : f_c/f_s = 1000/18000 ; n = 12
STEEL : TENSION = 18000 #/□"
COMPRESSION = $16000 - 4\left(\frac{L}{r}\right)^2$
COMPR. IN TOP CHORD = 12000 #/□" (COMPOSITE SECTION)
LOADING : H20 ; IMPACT = $\frac{50}{L+125}$ = < .30

WEIGHTS:

	TOTAL	PER LIN FT (L
CONCRETE DECK INCL. RAILING	472,000 LBS	3600 LB
SUPPORTING STEEL CONSTRUCTION	158,866 "	1211 "

QUANTITIES IN CONCRETE DECK:

ESTIMATED YARDAGE IN CONCR. DECK = 117 (CY)
" " REINF. STEEL = 3000 LB

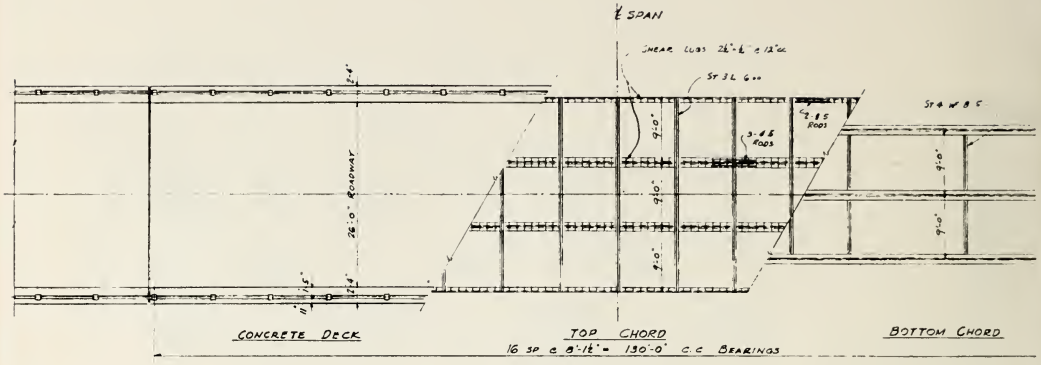


INTERIOR TRUSS Sc. 1"-1'-0"

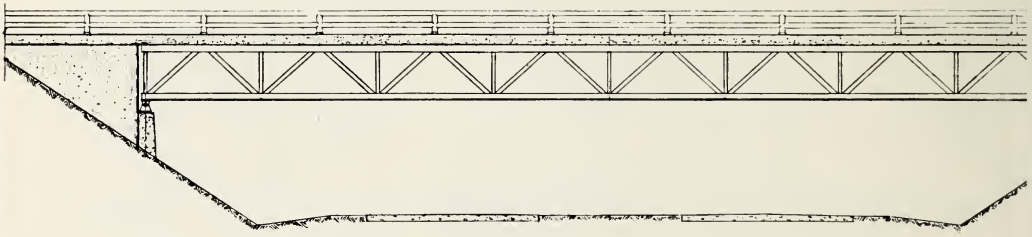
TRUSSES SIMILAR, SEE DESIGN, PAGE 16

OM CHORD

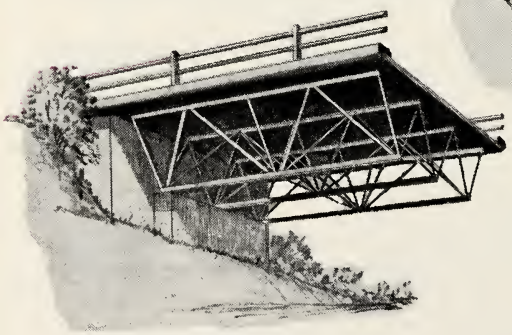
Figure 48



PLAN

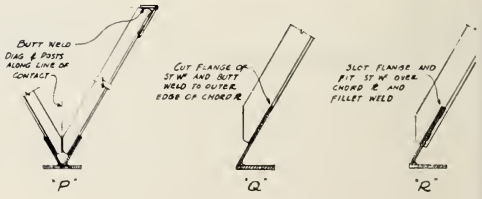


ELEVATION

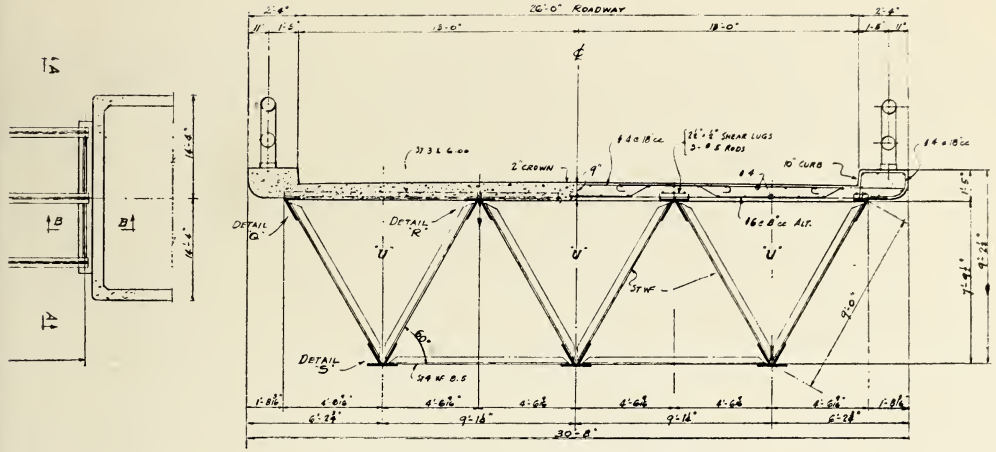


SKETCH I

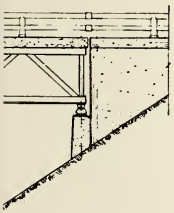
SKETCH II

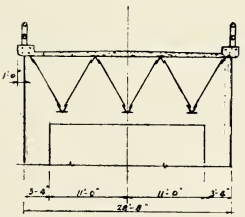

BUTT WELD DIAG & POSTS ALONG LINE OF CONTACT
CUT FLANGE OF ST W AND BUTT WELD TO OUTER EDGE OF CHORD R
SLOT FLANGE AND FIT ST W OVER CHORD R AND FILLET WELD
"P"
"Q"
"R"

ALTERNATES

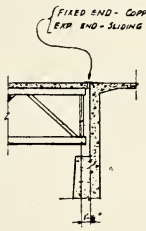


TYPICAL CROSS SECTION

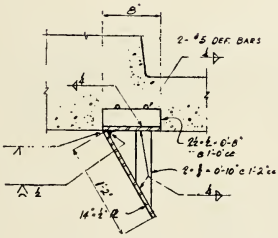




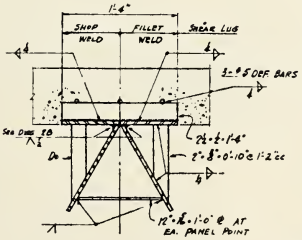
SECTION A-A



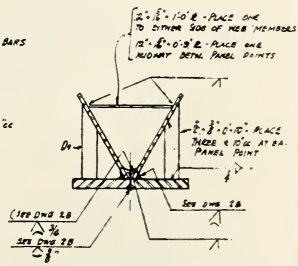
SECTION B-B



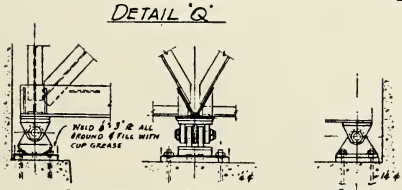
OUTER TOP CHORD
DETAIL 'Q'



INNER TOP CHORD
DETAIL 'R'


2" x ⅜ x 1'-0" R - PLACE ONE TO EITHER SIDE OF WEB MEMBERS
12" x ⅜ x 0'-9" R - PLACE ONE W/DIRT BETW. PANEL POINTS
2" x ⅜ x 0'-10" - PLACE THERE & 12" c/c AT EA. PANEL POINT
SEE DWG 28
SEE DWG 28

BOTTOM CHORD
DETAIL 'S'



EXPANSION BEARING

FIXED BEARING

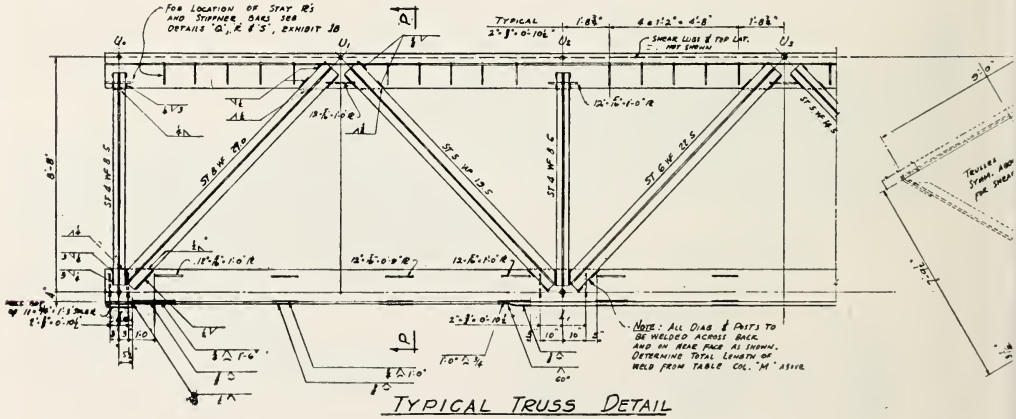
NOTE: BEARINGS TO BE DESIGNED FOR 150 KIPS, ALL WELDED CONSTRUCTION.

Figure 49

MEMBER	STRESSES PER TRUSS FROM PO ④ D.L. +	D.L. −	L.L. +	L.L. −	IMPACT +	IMPACT −	SUMMATION TENSION	SUMMATION COMPR.	DIMENSIONING l	r min	l/r	ALLOWABLE STRESS R/in²	AREA REQ'D	SHAPE USED	SECTION USED	AREA OF SECTION	WEIGHTS PER TRIANGULAR UNIT WT PER FT	No. OF PIECES	LENGTH OF EA.	WEIGHT IN POUNDS	SHOP OR FIELD WELD	DESIGN OF ALLOWABLE STRESS, R/in²	AREA REQ'D
L_0-L_1	82		31		10		83							L	8"s 14-# & 8-#	10.0	34.0	4	14.75	20.1	S	18	8·4/9
L_1-L_2	117		80		25		222							L	" " " 8-#	13.0	44.2	4	16.25	288	S	18	8·5/9
L_2-L_3	166		118		35		315							L	" " " 8-1/4	17.0	57.9	4	16.25	37.6	S	18	8·7/9 16·5/9
L_3-L_4	192		131		41		364							L	" " " 8-1/2	21.0	71.6	4	18.25	52.1	S	18	
																				138.7			
U_0-U_1														L	8"s 14-# & 8-#	10.0	34.0	4	8.62	11.7	S	18	
U_1-U_2		84		58		18		84				18	4.7	L	8"s 14-# & 8-#	10.0	34.0	4	16.25	11.1	S	18	
U_2-U_3		146		99		30		146				"	8.0	L	" " " 8-#	10.0	34.0	4	16.25	19.4	S	18	8·3/9
U_3-U_4		181		125		39		181				"	10.1	L	" " " 8-7/8	10.5	35.8	4	16.25	23.0	S	18	8·3 7/9 16·3/9
U_4-U_5		192		133		41		192				"	10.6	L	" " " 8-#	11.0	37.3	4	16.19	15.1	S	18	
																				91.6			
L_0-U_1		66		46		9		121	114	1.08	6	16.06	8.6	L	ST 8 WF	8.52	29.0	4	11.0	12.9	S	18·87=12.4	9.7
L_1-U_2		93	2	37	1	8		93	114	1.59	72	13.7	6.8	L	ST 6 WF	6.80	22.5	4	11.0	10.0	S	"	7.6
L_2-U_3		31	6	24	2	6		63	114	1.27	90	13.0	4.8	L	ST 5 WF	4.85	16.5	4	11.0	7.8	S	"	5.1
L_3-U_4		13	11	19	3	5	1	37	114	1.25	91	12.9	2.9	L	ST 5 WF	3.10	10.5	4	11.0	4.6	S	"	3.0
																				34.6			
L_1-U_1	57		39	1	9	-	104					18	5.8	L	ST 5 WF	5.76	19.5	4	11.0	8.6	S	"	5.6
L_2-U_2	90		30	6	7	1	77					18	4.3	L	ST 5 WF	4.27	14.5	4	11.0	6.4	S	"	4.2
L_3-U_3	22		72	8	5	2	89					18	2.7	L	ST 4 WF	2.96	10.0	4	11.0	4.4	S	"	2.9
L_4-U_4	5		16	13	2	2	25	10	116	1.13	100	(15-3)/15	1.5	L	ST 6 WF	2.50	8.5	4	8.0	3.7	S	"	2.0
																				23.1			
L_1-U_1		6		164		4		264	84	1.13	75	13.6	1.9	L	ST 4 WF	2.50	8.5	4	7.8	2.7	S	"	2.1
L_2-U_2		6		164		4		264	"	"	"	"	"	L	" " "	"	"	4	"	2.7	S	"	2.1
L_3-U_3		6		164		4		264	"	"	"	"	"	L	" " "	"	"	4	"	2.7	S	"	2.1
L_4-U_4		6		164		4		264	"	"	"	"	"	L	" " "	"	"	4	"	2.7	S	"	2.1
																				10.8			
TOP LAT							4		90	1.13	80	13.4	1.2	L	ST 4 WF	2.50	8.5	2	9.0	1.5	S	"	1.3
END PLATES L_0-U_0	END TRANSVERSE TRUSS (SEE PG 12)						63		90	1.13	80	11.9	3.1	L	ST 5 WF	3.10	10.5	4	7.8	3.3	S	"	2.5
BOTT LAT							16		90	1.13	80	13.6	1.2	L	ST 4 WF	2.50	8.5	4/3	8.5	1.0	F	"	1.3
																				5.8			
TOP LATERALS	INTERMEDIATE (SEE PG 11)						2.6					18	.15	L	ST 3 L	1.17	6.0	15	9.0	8.1	S	12.4	2
BOT "							23.2		90	1.13	80	13.4	1.76	L	ST 4 WF	2.50	8.5	14/3	8.5	9.0	F	"	1.9
																				12.1			
TOP OF EXC. WIDE STRIPS														BAR	2" × 7/8" · 0'-10"		2.5	244	.87	6.1	S		
TOP INT. RS														R	12"×7/16"·1'-0"		12.7	10	LIN. FT.	1.3	F		
BOT " "														"	"		12.7	19.35	" "	2.5	S		
																				9.9			
SHEAR LUGS														BAR	2"×1/2"	1.25	4.3	128	1.33	7.3	S		
PINS IN LUGS														BAR	7/8"ϕ (DEF.)	.36	1.2	10/3	131	5.2	S		
																				12.5			
LONG WELD TOP CHORD																					S		
" " BOT "																					S		
" " BOT "																							
SOLE R														R	11"×3/8"·1'-3"			2		7	S	.7	
BEARINGS																				15.0	S		
															ESTIMATED WT OF 1 UNIT = 354.0								
															TOTAL ESTIMATED WT = 354.0 · 3 · 100 = 106,440 LBS								

WEIGHTS:	TOTAL	PER LIN FT (BASED ON 131' LENGTH)
CONCRETE DECK INCL RAILING	472,000 LBS	3600 LBS
SUPPORTING STEEL CONSTRUCTION	106,440 LBS	813 LBS

ESTIMATED YARDAGE IN CONCR DECK =
" REINF. STEEL " " " =



TYPICAL TRUSS DETAIL

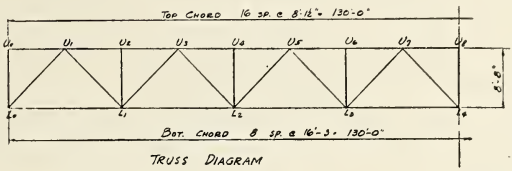
NOTE: TRUSS TO BE CAMBERED 3"

OF WELD (PER TRIANG. UNIT)

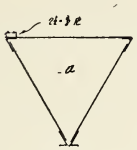
AREA REQ'D	SIZE USED	REQ'D LENGTH	LENGTH USED	TOTAL WELD LENGTH IN FT PER & UNIT	TYPE OF WELD
8 - 1/16	1/4	8	8	2.7	BUTT
8 - 3/16	3/16	8	8	2.7	"
8 - 1/16	1/16	8	8	2.7	"
14 - 1/2	1/2	14	14	4.7	"
8 - 1/8	1/8	8	8	2.7	BUTT
22 - 1/2	1/2	22	22	7.4	BUTT
9.7	1/2	27	27	18.0	FILL
7.6	3/8	27	27	18.0	"
5.1	5/16	23	23	15.4	"
3.6	1/4	17	18	12.0	"
1.6	1/4	24	24	16.0	FILL
4.2	3/8	23	23	15.4	"
3.9	5/16	18	18	12.0	"
2.0	1/4	11	18	12.0	"
2.1	1/4	12	18	12.0	FILL
2.1	1/4	12	18	12.0	"
2.1	1/4	12	18	12.0	"
2.1	1/4	12	18	12.0	"
1.3	1/4	7	7	2.3	FILL
5.5	1/4	14	18	12.0	"
1.3	1/4	7	7	1.6	"
.2	1/4	1	8	20.0	FILL
1.9	1/4	11	11	3.6	"
	1/4		12 Lgs.	244	FILL
	1/16			20	"
	1/4			39	"
	1/4		16 Lgs.	170	FILL
	1/4			31	FILL
	1/4		131.5	526	SPECIAL
	3/4		131.5	263	"
	5/16		131.5	263	"
	3/8			25	"
	1/2			8	FILL
	1/2			24.0	FILL
					BUTT

"M"

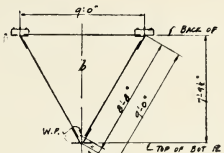
K = 117 cuyd. = 12000 LBS



TRUSS DIAGRAM



UNIT "a" - 2 REQ'D



UNIT "b" - 1 REQ'D

NOTES:
DESIGN IN ACCORDANCE WITH STD SPECS FOR WELDED HIGHWAY & RAILWAY BRIDGES DATED 1943 & STANDARD SPECS FOR HIGHWAY BRIDGES, 1941 EDITION.

$\ell/r = \angle 120$ FOR ALL MEMBERS

UNIT STRESSES USED:
CONCRETE: $F_c/F_s = 1000/18000$; $n = 12$.
STEEL: TENSION = 18000 #/□"
COMPRESSION = $15000 - 40\left(\frac{\ell}{r}\right)^2$
COMPR. IN TOP CHORD = 18000 #/□" (COMPOSITE SECTION)

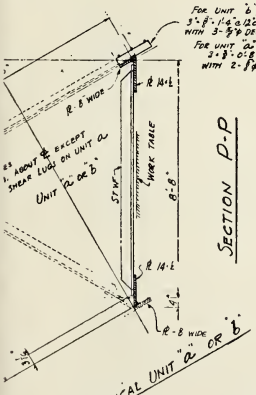
LOADING = H20
IMPACT = $\frac{50}{L+125} = \angle .30$

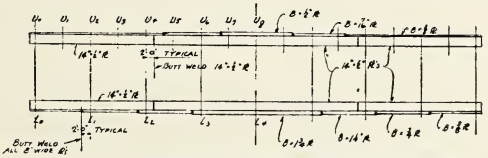
SUMMATION OF WELDS (FOR TABULATION OF COSTS SEE DESIGN, SHT ⑰)

	BUTT WELDS - SHOP						FILLET WELDS - SHOP				FILLET WELD FIELD		SPECIAL WELDS - SHOP					
	3/8	7/16	1/2	3/4	1/4	1 1/2	1/2	9/16	3/8	1/4	1/4		1/2	1/2	3/4	3/4	3/8	
1 UNIT LIN FT	5.4		12.1	26.7	2.7		590.0	27.4	33.4	43.0	30.2		524	386	138	29	19	
3 UNITS LIN FT	16.2		36.3	80.1	8.1		1770.0	82.2	100.2	129.0	90.6		1572	1158	414	87	57	
WEIGHT OF WELD PER LIN. FT	.55		90	1.90	3.40		.22	.31	.37	.57	.44		.35	70	1.20	1.30	.40	TOTAL WT (LBS)
TOTAL WEIGHT OF WELD (SUM'D)	9		33	152	28		390	26	37	74	40		550	810	538	113	35	2795 SHOP & 40 FIELD

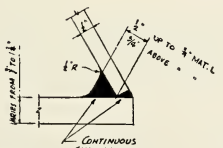
WEIGHT OF SHOP WELD PER TON OF STEEL = 2795/53.2 = 52.5 LBS
" " FIELD " " " " = 40/53.2 = .8 LBS

* BASED ON THE ASSUMPTION THAT EACH TRIANG. UNIT IS COMPLETELY SHOP FABRICATED, TRANSPORTED & ERECTED AS ONE PIECE.

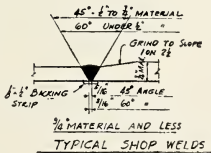




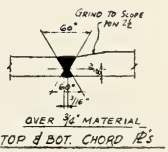
ARRANGEMENT OF TOP & BOT. CHORD BUTT WELDS
(NO SCALE)



TOP OR BOT. CHORD ANGULAR SECTION

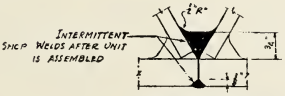


3/16" MATERIAL AND LESS



OVER 3/16" MATERIAL

TYPICAL SHOP WELDS OF TOP & BOT. CHORD ℄'S



INTERMITTENT WELDS FOR BOTT. CHORD

Figure 50

"The design submitted shows the posts and diagonals fillet welded to the outstanding leg of the top and bottom chord plates. This, in the author's opinion, is the most practical and economical method and is comparable to vertical trusses having angles as web members. While approximately two-thirds of the direct stress passes in almost a straight line through the flange of the T sections between the top and bottom chord plates, a slight eccentricity exists, which may be objectionable.

"Figure 49 shows three methods of what might be done to further reduce eccentricities of the inclined truss members:

"Alternate 'P': The web members of the two adjacent trusses are butt welded to each other at the bottom chord along these lines of contact. At the top, a portion of the web of the inclined members is welded to the underside of the horizontal chord plate.

"Alternate 'Q': The flanges of the web members are cut to fit the outstanding edge of the chord plates and then butt welded to it. The extending portion of the web is then welded to the 14 x ½ in. plate of the top and bottom chords.

"Alternate 'R': The webs of inclined web members are slotted to fit over the 14 x ½ in. plates of the top and bottom chords. This will require a milling operation since the entire fillet must be removed flush to the face of the flange. The member then is fitted over the chord plates and welded to it.

"Of the three alternates shown, the author prefers method P since it eliminates all eccentricities and will cost the least. While all three alternates mean an increase in fabricating cost, it is pointed out that such an increase will affect the advantages of the welded design only slightly.

"The open angle of the top chord section is provided with shop welded vertical stiffeners at 14 in. centers. In addition a horizontal stay plate, field welded, is placed between the inclined chord plates at every panel point.

"The bottom chord is provided with three stiffeners on the outer side of each panel point and with four stay plates each, between the panel points. All stiffeners and stay plates for the bottom chords are shop welded.

"Because the deck construction is shallow in depth and violates the ratio of depth requirements as set forth in the Highway Specifications Section 3, 6 and 11, the deflections were thoroughly checked. The ratio of depth to the length of span, including the depth of the deck slab and curb, is $\frac{9.35}{130.0} = \frac{1}{13.8}$

"However, the depth ratio, according to specification requirements, should be L/10, which would require a depth of 13 ft. As computed in the design a truss with a required depth of 13 ft. would deflect approximately 2¼ in. (assuming no composite action). Investigations as to deflection of the submitted composite design show the following results:

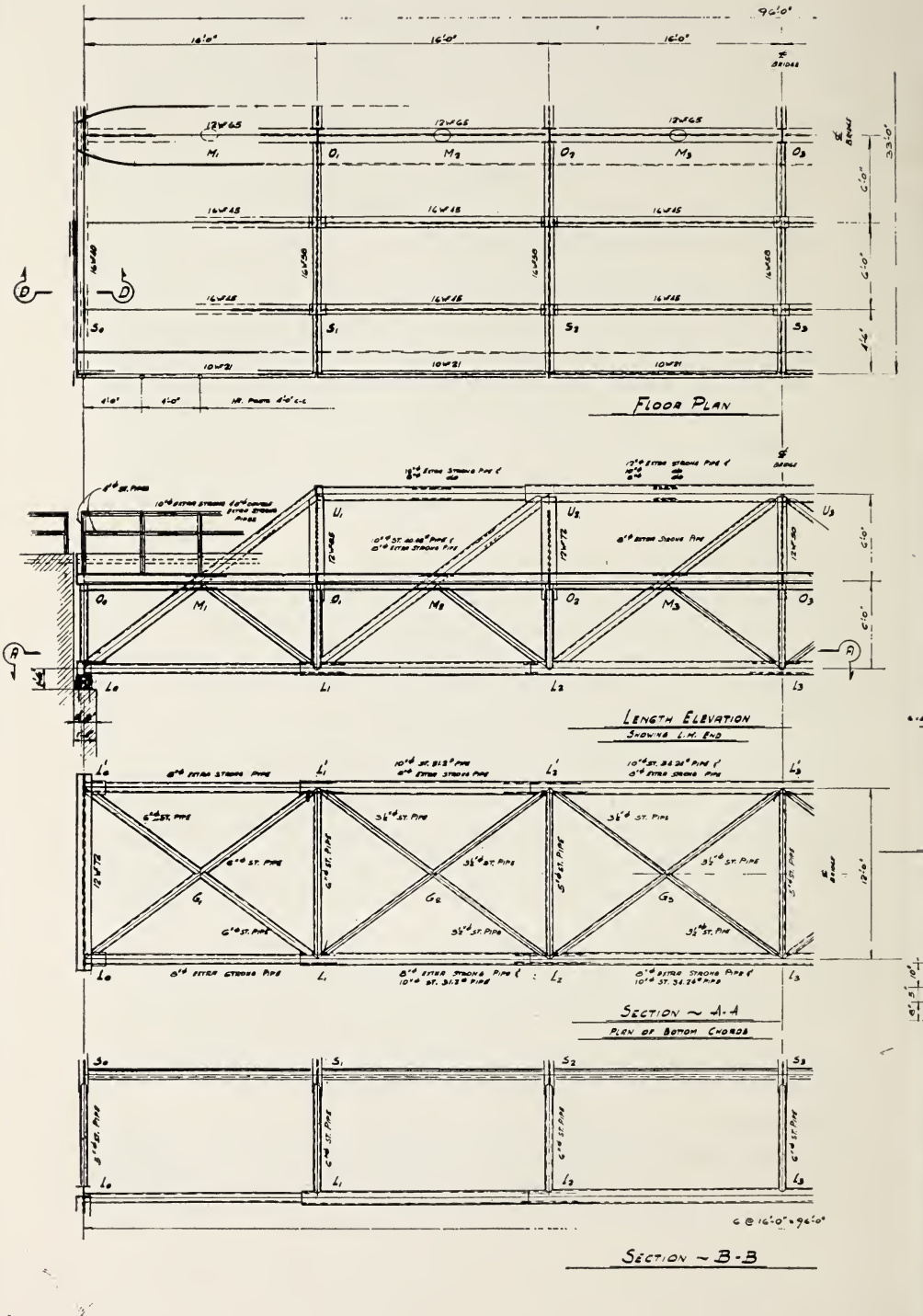
Approximate Deflection by Moment of Inertia Method . . 2⅛ in.
Approximate Deflection by Top and Bottom
Chord Average—Unit Stresses . . . . . 2⅛ in.
Correct Deflection by Work Method . . . . . 2⅝ in.
Correct Deflection by Williot Diagram . . . . . 2½ in.

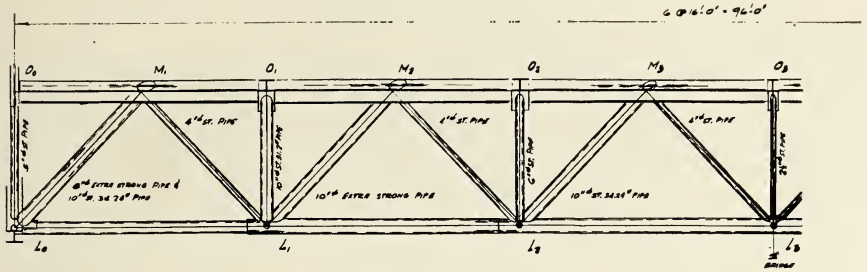
"From these investigations, it may be concluded that no excessive deflection occurs and that sections need not be increased to reduce deflection. The main factor causing low deflection with lesser depth is, of course, the large moment of inertia which is obtained from the composite section. Of 2⅝ in. total deflection, 1¾ in. is due to Dead Load, and ⅞ in. is due to Live Load; the latter is approximately $\frac{1}{1800}$ of the span. It is recommended that the trusses be cambered 3 inches.

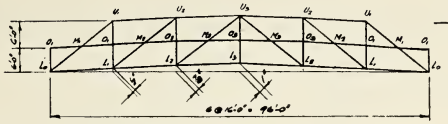
"The trusses may be fabricated in section or to full length in one piece, then shipped to the site, assembled into a triangular section and hoisted into position; or, they may be fabricated into a full triangular unit in the shop and transported to and erected at the site. This procedure would depend on location of site, access to it, and also on shop, clearance and transport conditions. To avoid undue stresses in the steel of the top chord while the concrete is freshly poured and still in a plastic condition, it is proposed to support the span at its third points temporarily. After hardening of the concrete, these supports are to be removed and the resulting compressive stresses will then largely be taken by fully engaging the concrete deck slab."

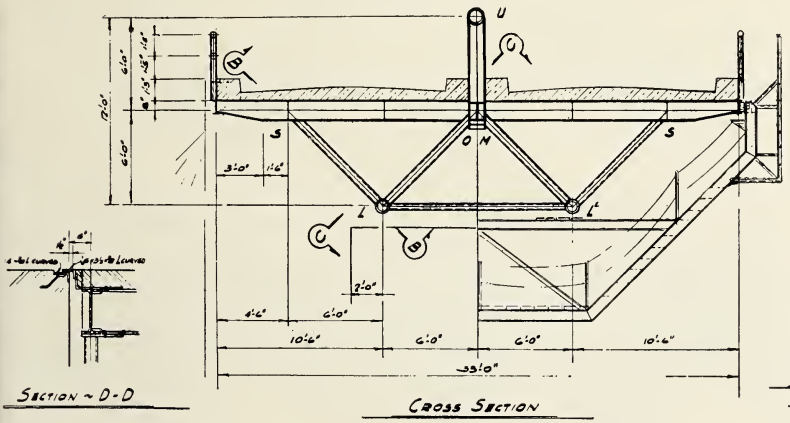
The weights and estimated costs are: Exhibit A, 158,844 lb. at $0.147 per lb. for a total of $23,400; and Exhibit B, 106,440 lb. at $.167 per lb. for a total of $17,800. For each, a total of 117 cu. yd. of concrete and 19,000 lb. of reinforcement bars are required. The welded structure requires 2795 lb. of shop welds and 40 lb. of field welds.

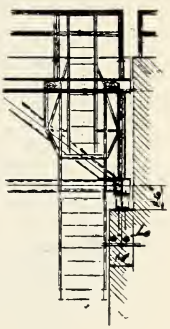
Franz Schulze, Evanston, Illinois and E. A. Bartkus, Chicago, Illinois designed the bridge shown in Figure 51. The bridge has six 16 ft. panels for a total length of 96 ft. Two 13 ft. roadways are separated by the upper part of the trussed structure. This single

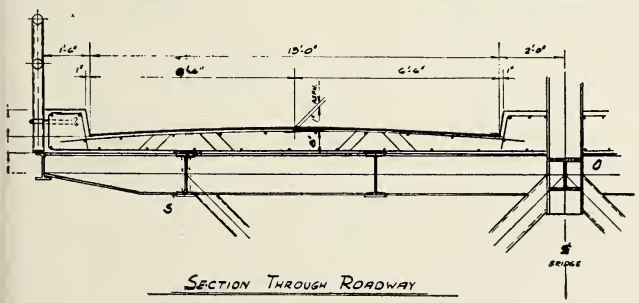

96'-0"
16'-0"
16'-0"
16'-0"
℄ BRGS
12WF65
16WF45
16WF36
16WF50
10WF21
33'-0"
6'-0"
6'-0"
4'-6"
4'-0"
4'-0"
HP. Posts 4'-0" c.c.
FLOOR PLAN
LENGTH ELEVATION
SHOWING L.H. END
SECTION ~ A-A
PLAN OF BOTTOM CHORDS
SECTION ~ B-B
6 @ 16'-0" = 96'-0"





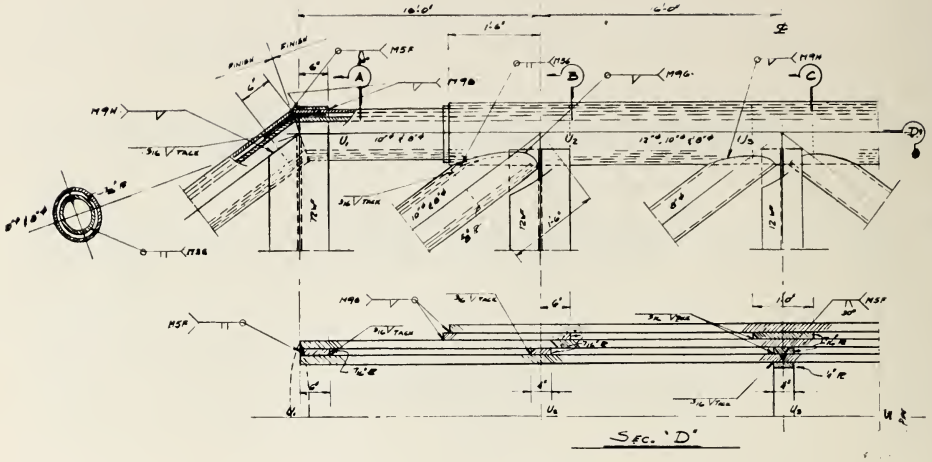






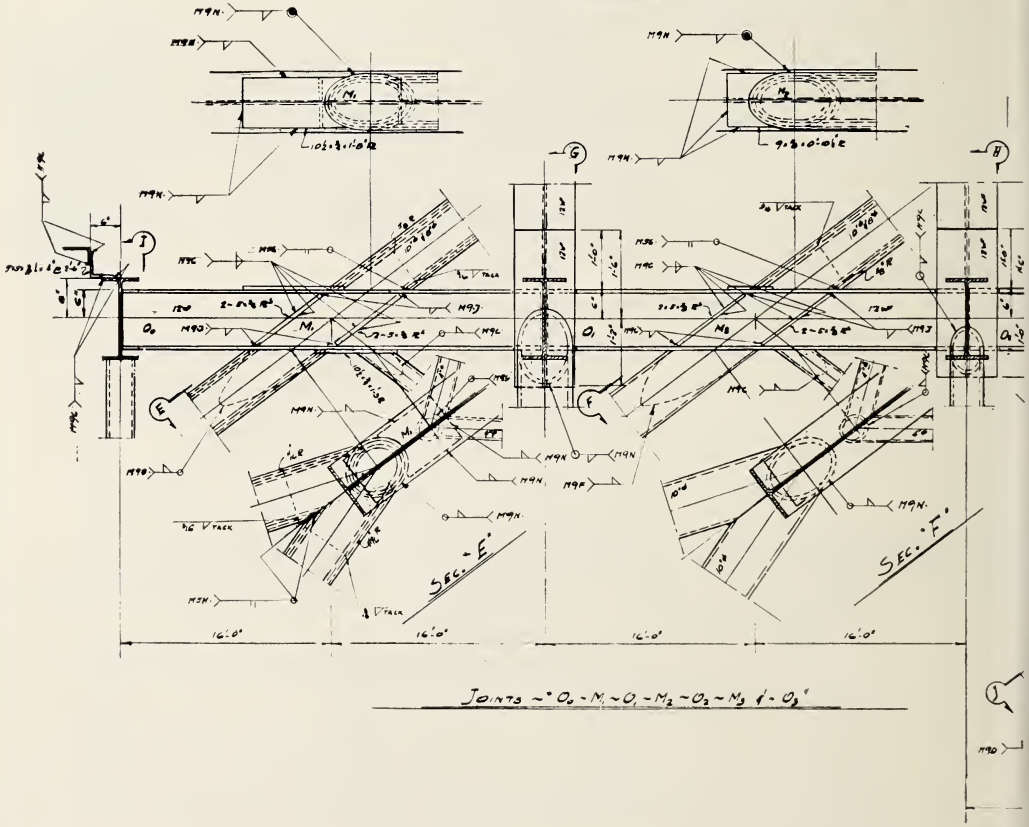
MATERIAL ~ STEEL AS PER ASTM - A7-46
WELDING ~ DESIGNED FOR THE SAME
LOADING ~ H20 - S16 - 44
PAINT ~ SHOP ONE COAT OF RED LEAD (WELDING AREAS NOT TO BE SHOP PAINTED)
~ FIELD TWO COATS OF GRAPHITE BRIDGE PAINT

Figure 51



SEC. 'D'

JOINTS ~ 'U₁ ~ U₂ & ~ U₃'



JOINTS ~ 'O₀ ~ M₁ ~ O₁ ~ M₂ ~ O₂ ~ M₃ & ~ O₃'

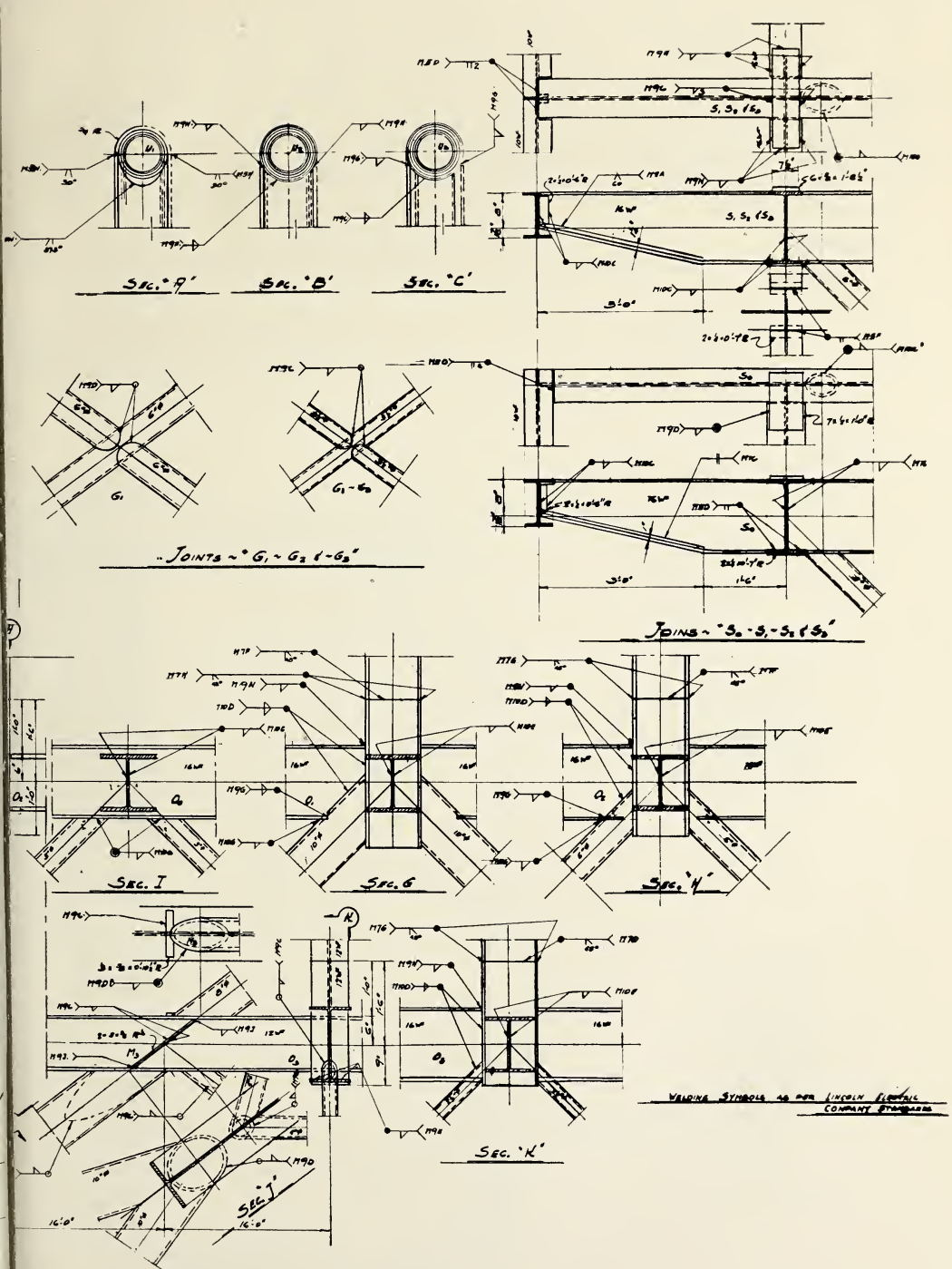
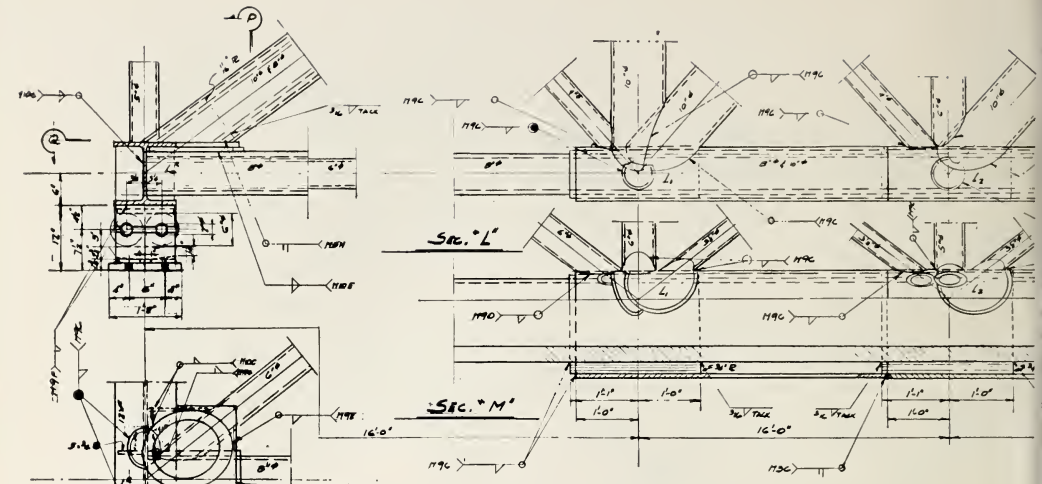
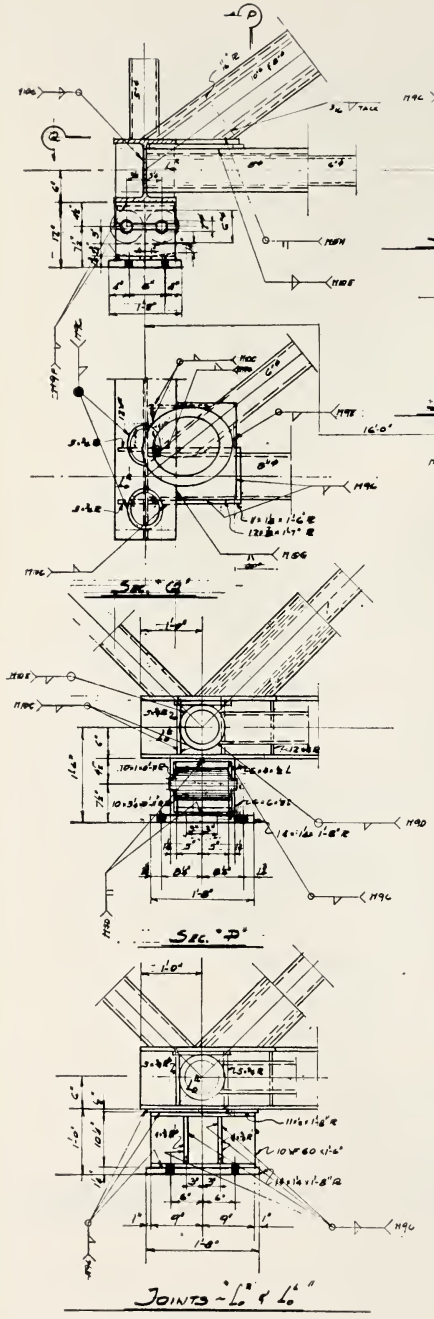

SEC. "A"
SEC. "B"
SEC. "C"
JOINTS ~ "G, ~ G2 & ~G3"
JOINS ~ "S0 - S1 - S2 & S3"
SEC. I
SEC. 6
SEC. "H"
SEC. "K"
SEC. "J"
WELDING SYMBOLS AS PER LINCOLN ELECTRIC COMPANY STANDARDS

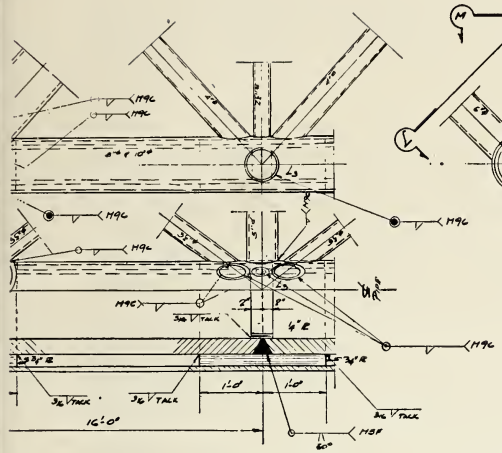
Figure 52

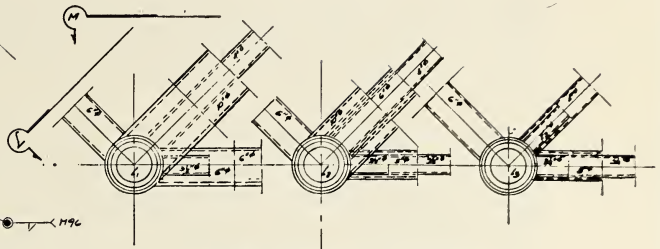


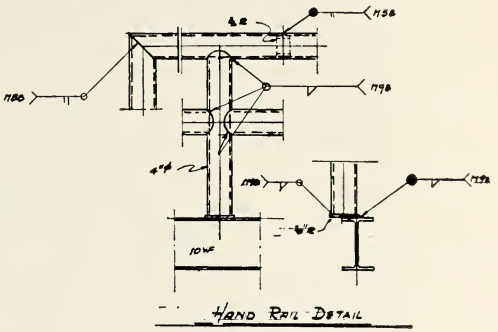


COMPARISON TABLE

	EXHIBIT "A" ~ RIVETED STRUCTURE							EXHIBIT "B"		
	MEMBER SECTION	WT. lbs per ft.	LENGHT regd.	No regd.	WEIGHTS	SUB-TOTALS	lbs per ft. GRAND	MEMBER SECTION	WT. lbs per ft.	LENGHT regd.
SUPPORTING MEMBERS ~ (TRUSS)	12 WF	133.0	100	2	2,660			8"ø D.B.Iron Pipe	72.42	100'
	12 WF	106.0	7.0	2	1,480			12"ø E. Std. Pipe	68.62	35
	12 WF	92.0	12.5	2	2,300			10"ø do	54.74	32.2
	12 WF	85.0	9.0	2	1,530			10"ø do	do	11.0
	12 WF	do	7.0	2	1,190			10"ø do	do	10.0
	12 WF	66.0	7.0	2	910			8"ø do	43.39	32.2
	12 WF	49.0	9.0	2	810			8"ø do	do	32.0
	15 C	40.0	34.0	4	5,440			8"ø do	do	9.2
	15 C	do	66.0	4'	12,800			8"ø do	do	9.0
	15 C	33.9	16.5	8	4,270			8"ø do	do	11.0
	23 x ⅜ PL	68.7	18.0	2	2,110			10"ø ST. PIPE	40.48	9.2
	23 x ⅜ PL	do	5.0	1	290			10"ø do	34.24	35.0
	22 x ⅝ PL	23.4	34.0	2	1,590			10"ø do	do	11.0
	8 x 4 x ½ L	37.4	9.0	8	2,690			10"ø do	31.20	14.0
	8 x 4 x ½ L	26.2	6.75	8	1,360			10"ø do	do	7.75
	7 x 4 x ⅜ L	30.2	8.75	8	2,110			6"ø do	18.97	11.3
	6 x 4 x ⅝ L	18.1	8.75	8	1,270			6"ø do	do	19.3
	5 x 5 x ⅜ L	13.3	6.75	12	1,000			6"ø do	do	9.75
	do	do	11.0	10	1,350			6"ø do	do	9.5
	do	do	8.0	8	790			6"ø do	do	7.75
	do	do	17.86	6	880			5"ø do	14.61	11.3
	5 x 3½ x ⅝ L	17.0	9.0	24	3,590			5"ø do	do	7.5
	6 x 3½ x ⅝ L	7.7	8.0	16	990			4"ø do	10.79	11.0
	do	do	11.75	8	1,100			3½"ø do	9.11	19.6
	6 x 3 x ⅝ L	7.2	6.5	8	370			3½"ø do	9.8	8.6
	SKETCH R°				31,050			2½"ø do	5.79	7.5
	RIVETS				2,980			12 WF	58.0	7.2
	SHOP PAINT				650			12 WF	72.0	7.0
						88,810	980	12 WF	72.0	10.0
								12 WF	50.0	7.0
								SKETCH R°		
								WELD ELECTRODES		
								SHOP PAINT		
DECK MEMBERS ~ (FLOOR)	16 WF	71	16.0	10	11,350			12 WF	60.0	16.0
	16 WF	58	16.0	24	22,300			16 WF	58.0	16.2
	16 WF	50	38.0	2	2,300			16 WF	45.0	16.1
	10 WF	21	16.5	6	1,380			16 WF	40.3	33.1
	10 WF	21	16.0	8	2,690			10 WF	21.0	16.2
	12 C	25	16.0	12	4,800			10 WF	21.0	16.
	4 x 3½ x ⅜ L	9.1	7	20	1,280			5"ø ST. PIPE	14.62	7.5
	4 x 3 x ⅜ L	7.2	7	8	400			6"ø do	18.97	7.5
	SKETCH R° & Cpl.				2,580			SKETCH R°		
	RIVETS				1,020			WELD ELECTRODES		
	SHOP PAINT				240			SHOP PAINT		
						50,360	624			
MISC. RD. WAY & SIDE WALK BRACING	GC	8.2	384		3,120			4"ø ST. PIPE	10.79	479
	4 I	7.7	200		1,540			6 x 3½ x ½ L	11.7	66
	6 x 3½ x ½ L	11.7	66		770			WELD ELECTRODES		
	RIVETS				290			F.P. BEARINGS		
	BEARINGS				2,000			F.P. BEARINGS		
	SHOP PAINT				40			SHOP PAINT		
						7,760	81			
TOTAL STEEL WEIGHT						146,610	1,525			
8" R.F. CONCRETE DECK WEIGHT						336,000	3,500			
GRAND TOTAL BRIDGE						482,610	5,025			







SCALE ~ 1" = 1'-0" & NOT TO SCALE

OF BRIDGES

"B" ~ WELDED STRUCTURE — COMPARATIVE SAVING

Length req'd.	No. req'd.	Weights	Sub Totals	lbs. per ft. Bridge	Total Wt. lbs.	lbs. per ft. Bridge	Percentage
10.0'	2	1,450					
35	1	2,290					
32.2	2	3,520					
11.0	4	2,600					
10.0	2	1,090					
32.2	2	2,790					
32.0	6	9,320					
9.2	2	800					
9.0	2	770					
11.0	2	1,910					
9.2	2	750					
35.0	2	2,600					
11.0	8	3,020					
16.0	4	2,000					
7.75	4	950					
11.3	2	440					
19.3	2	730					
9.75	2	370					
9.5	2	360					
7.75	4	590					
11.3	3	500					
7.5	4	460					
11.0	12	1,620					
19.0	4	690					
8.0	8	690					
7.5	2	90					
7.25	2	1,230					
7.0	2	1,010					
14.0	2	2,020					
7.0	2	350					
		1,870					
		486					
		240					
			49,980	500	40,530	420	15.6%
16.0	6	6,240					
16.0	10	9,260					
16.0	24	17,300					
33.0	2	2,640					
16.3	4	1,380					
16.0	8	2,690					
7.5	4	440					
7.5	10	1,420					
		580					
		115					
		210					
			42,270	440	8,070	84	14.0%
479		5,160					
66		770					
		86					
		1,240					
		440					
		40					
			7,730	80.5	30	.5	0%
			97,980	1020.5	48,630	504.5	33.3%
			336,000	3,500			
			433,980	4520.5	48,630	504.5	10%

FOR BENEFIT OF STRESSES

WELDING TABLE

Weld Type	lbs. of Elec. per ft.	ft. Weld Shop	ft. Weld Field	lbs. of Electrode Shop	lbs. of Electrode Field
M1C	.31	12.0	-	3.72	-
M7A	.62	30.0	-	18.60	-
M5B	.82	-	11.8	-	6.0
M9D	.85	5.35	7.35	4.55	6.25
M5F	1.50	65.50	23.40	85.00	30.50
M6G	1.71	49.33	-	76.08	-
M5H	2.11	31.50	-	56.60	-
M7D	.50	-	1.0	-	.50
M7F	.85	-	4.0	-	3.40
M7G	1.30	-	5.33	-	6.95
M7H	1.60	-	4.90	-	7.20
M9B	.16	503.6	-	80.66	-
M9C	.20	341.5	52.0	68.20	10.40
M9D	.30	16.0	33.0	4.80	9.90
M9E	.40	16.7	-	6.70	-
M9F	.45	19.0	-	7.55	-
M9G	.54	32.9	-	17.80	-
M9H	.70	11.7	-	8.20	-
M9J	1.10	19.2	-	21.20	-
M9N	.69	59.4	59.2	41.00	41.00
M10B	.14	-	7.7	-	1.08
M10C	.17	63.5	176.0	10.80	30.00
M10D	.26	-	16.7	-	4.35
M10E	.38	44.0	-	16.80	-
M10G	.67	1.0	1.0	.67	.67
TOTAL ELECTRODES WEIGHT				526.7 lbs.	158.2 lbs.
GRAND TOTAL				684.9 lbs.	

10.75 lbs Electrode Basis Value @ $.23 lbs. Electrode Field Value

Figure 53

upper chord, a pipe section, is connected below the roadway to two lower chords—also pipe sections. The bottom laterals and the inclined diagonals of this structure are also pipes. As the designers said:

"The design is based on the use of two lower half trusses inclined toward one another and joining at deck level into one upper half truss which extends above the bridge deck and thus serves as a traffic separator. The two lower chords, spread apart, are diagonally braced and form, together with a common deck stringer, a triangular space frame, making further bracing unnecessary while still giving ample torsional resistance. To this core of the structure are joined simple struts resting on the lower chords and supporting the outer sides of the cross beams and through them the deck stringers. Struts and frame present a cross section of three triangles blending nicely with the appearance of the length view of the bridge. The bridge deck is of reinforced concrete with curbed sidewalks on the outsides of the bridge and on both sides of the upper truss half in the length center of the bridge. The cross beams and stringers are of rolled sections. The deck, its elevation being between those of the upper and lower truss chords, will take the main part of the lateral (wind) stresses. The vertical posts above the deck are of sufficiently strong rolled section to withstand the lateral stresses occurring in the upper chord.

"The fabrication of the present type of design includes the cutting of pipe members in a certain way. Their ends join other members with cross sectional planes at an inclination to their length axis. At the center deck stringer, some of their junctions require two cuts in such angular inclination. The angles of these cuts are easily ascertainable and the cuts made accordingly. Holding plates are provided, which aid the proper placing of the members to be ready for being welded into the frame. Butt welding is being used on most of the pipe sectional members, fillet welds otherwise. To provide the needed larger areas for the respectively high stresses the outside pipe sections are reinforced with smaller diameter pipe sections. They are centered to each other by rings welded around the smaller pipe at all points of stress intersection. At the ends of each member the several pipes are welded together in such a manner that the combined sections come to full bearing. Aside of that the pipe members are to be reliably sealed at the ends to prevent access of air to the inside. Preceding this the pipes are completely dried by the use of appropriate chemicals and pressure tested so as to guard against possible development of inside rusting.

"The shop work is intended to complete the entire triangular frame, including the middle length beam at deck level and the stump lengths of the vertical posts of the upper half-truss. These stumps will serve an easier erection of the upper half-truss in the field.

"The shipping of the bridge is done in parts, but the whole triangular frame possibly in one piece, loaded on two railroad flat cars with the deckline tilted at 45 degrees to the horizontal. The upper half-truss is likewise shop completed and, like the struts and beams, can be shipped right alongside.

"The erection in the field thus requires putting the upper half-truss in place, its vertical posts being easily fastened to the stumps of the space frame, its diagonal members guided by holding plates, and then completely welded to the frame so as to cause the whole truss of the bridge to carry its full weight. This can be done on shore before moving the structure into place or—by means of scaffolding suspended from the lower chords of the space frame—on the final site. The latter procedure is advantageous insofar as a considerably lighter weight needs to be moved. This is favored by the fact that the bearings of the bridge fixed on one end and on expansion rollers on the other, are parts of the space frame. Easily accessible welding can then proceed to the point where the deck is ready for the application of the reinforced concrete slab. It may be mentioned with regard to this space frame that it is capable of carrying the erection loads while the upper half truss is not yet connected to it. However, the latter must be erected and thus the entire truss completed, before the concrete floor of the bridge deck is laid.

"The type of design was chosen particularly for the following reasons:

1. While it is not new (it has been used in Europe) it has not yet been built in America according to available information.
2. The criterion of usefulness being more or less the same for different countries, it is assumed that this type may be appropriate anywhere.
3. The advantages then are considered to be:
   a. The separation of direction of travel as this expedites traffic and makes for greater safety.
   b. Unobstructed vision toward the outside of the bridge.
   c. Protection against glare from the lights of approaching vehicles.
   d. Production of this type is considered advantageous as to

weight and cost, because of the need of fewer supporting members."

Details of the welded bridge are shown in Figures 52 and 53. The riveted bridge has the same shape and dimensions as the welded one but the members are composed of WF sections, channels, angles, and plates rather than pipes. The reinforced concrete requirement is identical. The riveted steel amounts to 146,610 lb.—the welded, 97,980 lb. The shop welds weigh 527 lb. and the field welds weigh 158 lb.

S. M. Bagdoyan, Washington, D. C., chose an unusual structure for his 80 ft. span. As shown in Figures 54 and 55, the bridge has a single vertical truss located between the two 24 ft. roadways. The through truss has a curved top chord and has a depth at mid-span of 11 ft. The top chord consists of a 26 in. cover plate and two 18 in. channels. The bottom chord is a cylinder with a wall thickness of 1 in. and a diameter of 36 in. The verticals and diagonals are 16 WF sections.

The reinforced concrete slab is supported at each panel point (every 10 ft.) by 28 ft. cantilever beams welded to both sides of the bottom chord of the truss. Mr. Bagdoyan explained the action of his single truss as follows:

"The main feature of this design is the use of a pipe as the bottom chord of the truss. Under the dead load alone the truss will sustain only balanced vertical loads. Whereas, the live load on two lanes only on either side of the truss will produce torsional stresses of great magnitude, which will be transmitted to the abutments through the pipe. The force couple resisting this torsion is provided by anchoring the ends of the cantilever beams into the abutments at both ends of the bridge.

"The top chord and web members of the truss were designed for maximum stresses produced by loading four lanes of the roadway simultaneously. The bottom chord was designed for combined tension and torsional shear. The concrete safety curb at the end of the cantilever beams is securely anchored to the steel framing in order to prevent the excessive deflection of each individual beam under concentrated wheel loads.

"Deflection at the center of the truss due to dead load is 0.86 in.

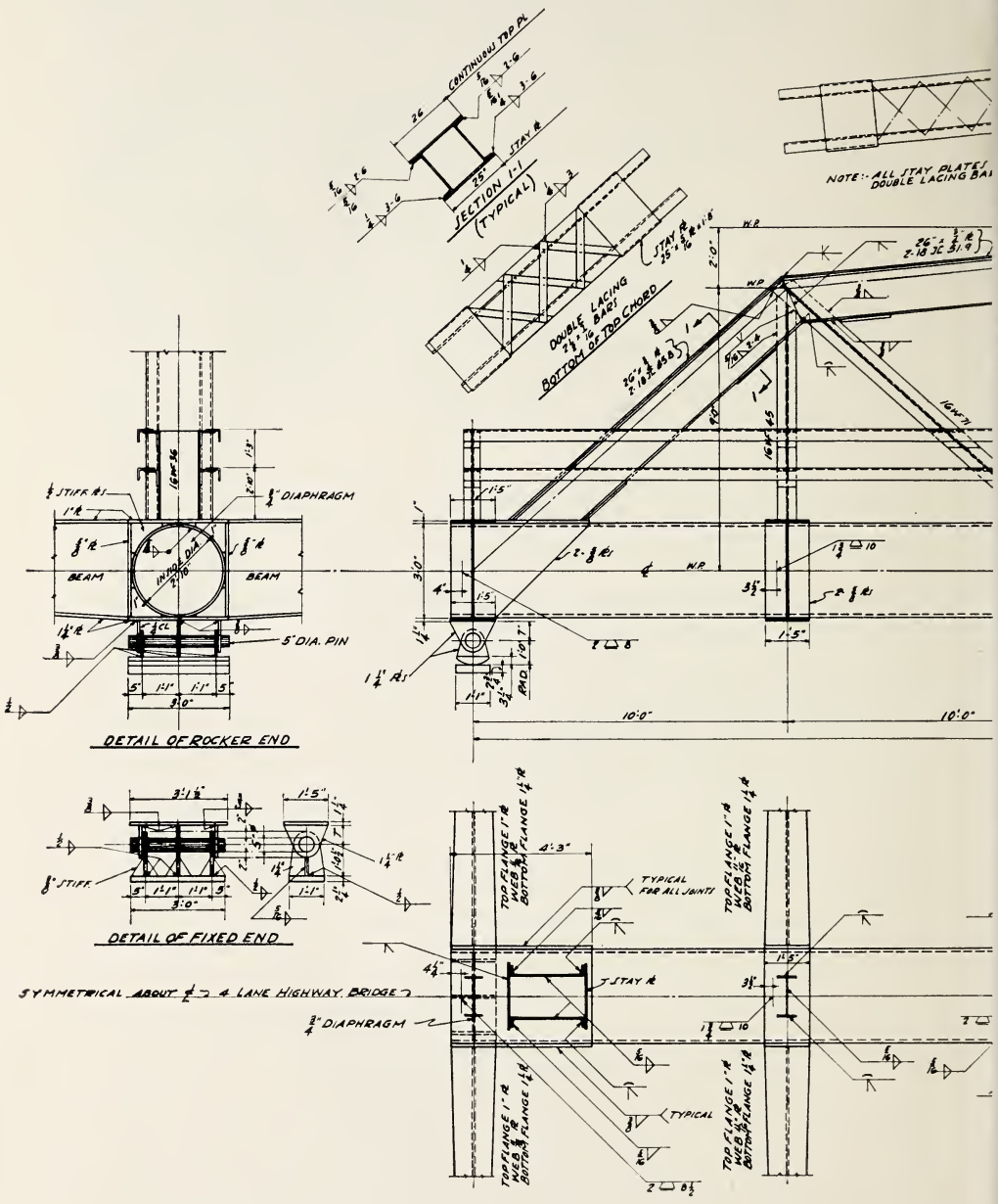
"Deflection at the end of the cantilever beams due to dead load only is 0.75 in.

"Deflection at the end of the middle cantilever beam due to bending moment and twist in the pipe produced by H15-S12-44 live loading is 0.97 in."

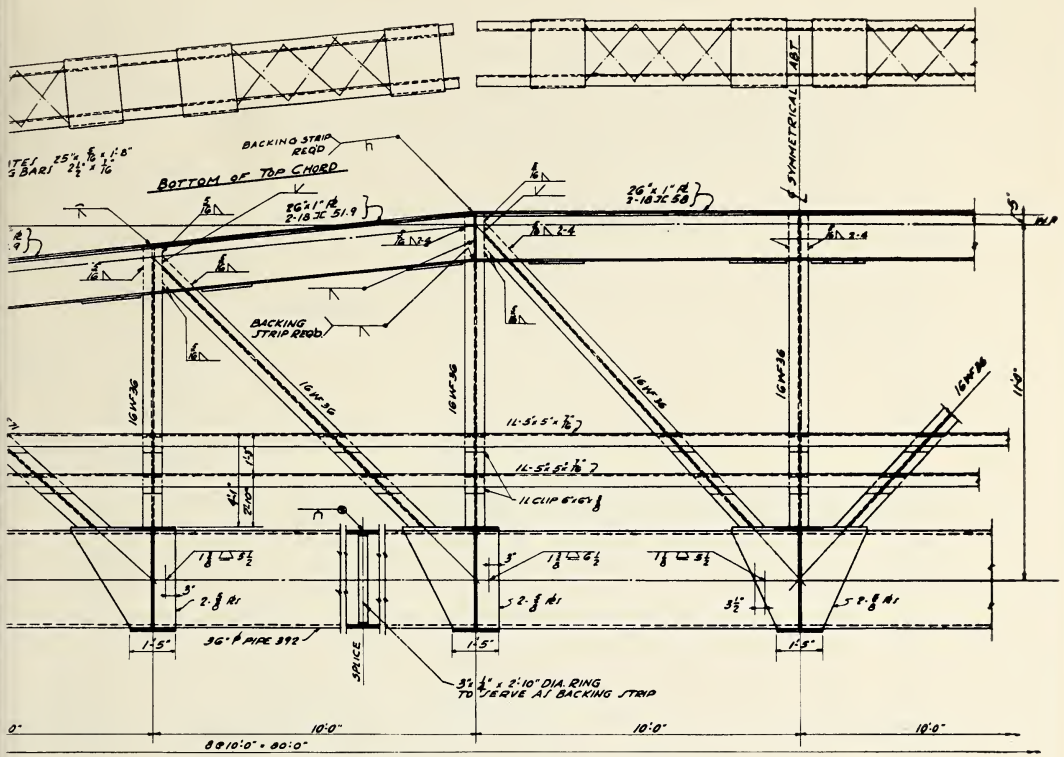
The details of the method utilized to prevent rotation at the abutments are given in Figure 55.

The following summary of quantities and costs compares the riveted and welded bridges.

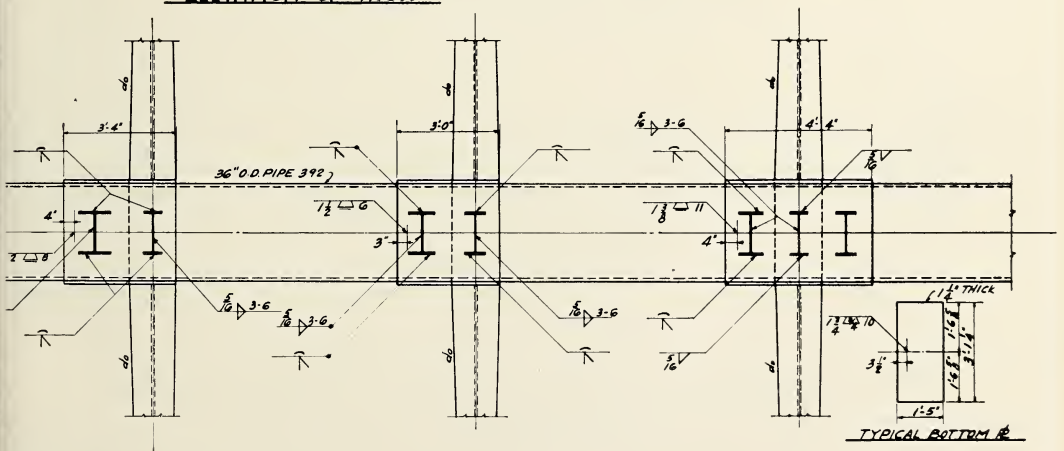
	Exhibit A	Exhibit B
Weight of steel	101.18 Tons	76.03 Tons
Surface to be painted	11,000 sq. ft.	7,500 sq. ft.
Concrete	140.1 cu. yd.	142.3 cu. yd.
Reinforcing steel	24,550 lb.	25,284 lb.


CONTINUOUS TOP PL.
STAY PL.
SECTION 1-1 (TYPICAL)
DOUBLE LACING BARS
BOTTOM OF TOP CHORD
NOTE: ALL STAY PLATES DOUBLE LACING BA
W.P.
W.P.
DIAPHRAGM
STIFF. PL.
BEAM
BEAM
5" DIA. PIN
DETAIL OF ROCKER END
DETAIL OF FIXED END
STIFF.
SYMMETRICAL ABOUT ℄ — 4 LANE HIGHWAY BRIDGE
DIAPHRAGM
TOP FLANGE
WEB
BOTTOM FLANGE
TYPICAL FOR ALL JOINTS
STAY PL.
TYPICAL
10'-0"
10'-0"

BOTT

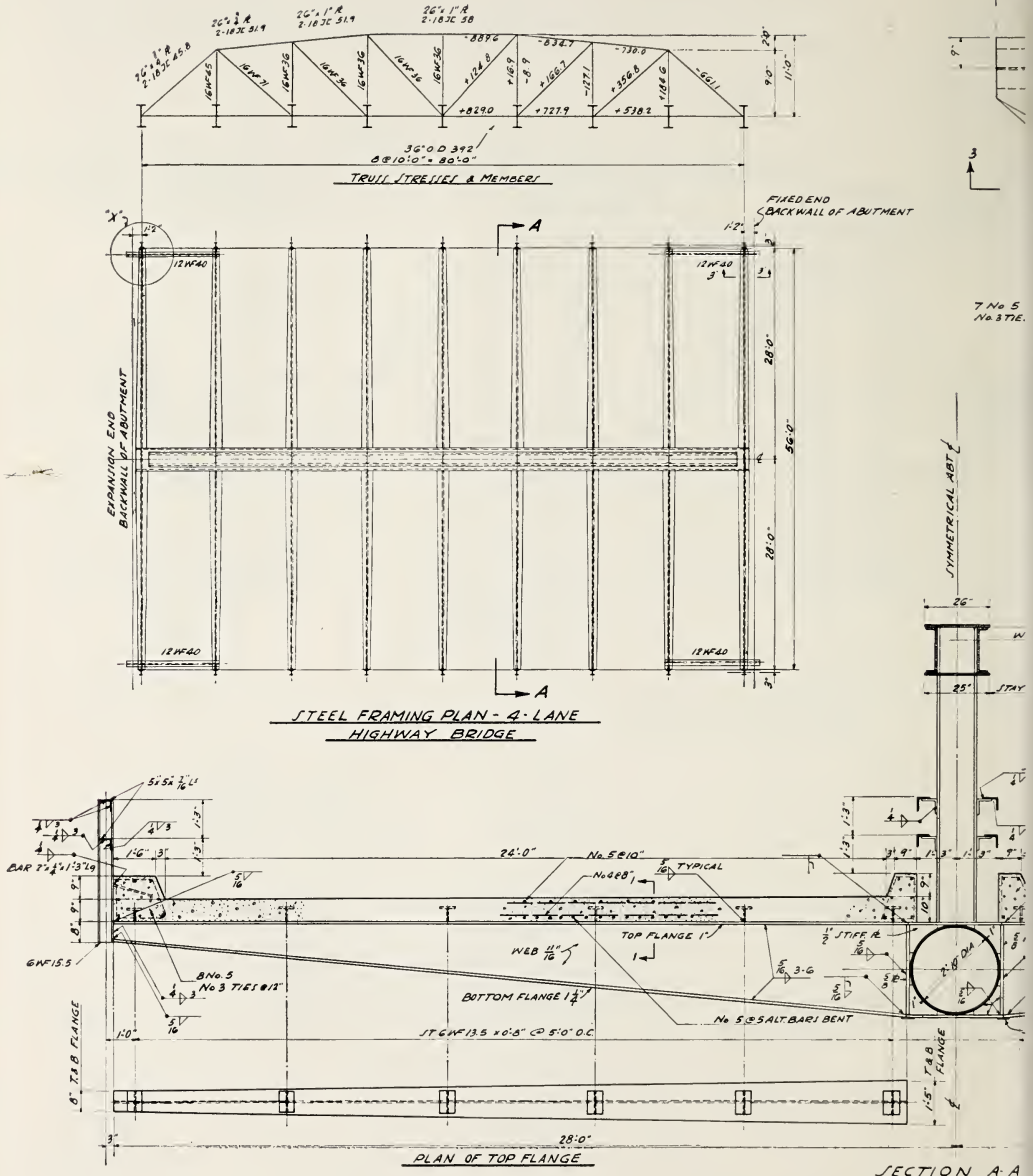


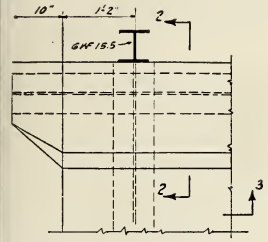
ELEVATION OF TRUSS



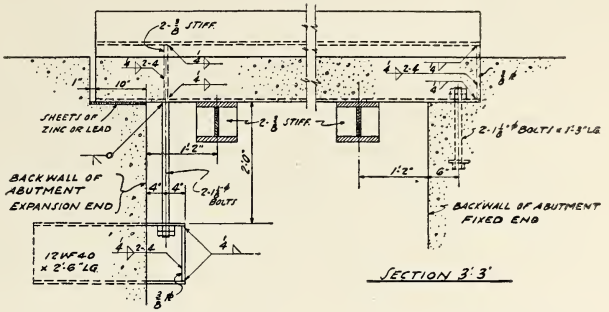
BOTTOM CHORD FRAMING PLAN

Figure 54


TRUSS STRESSES & MEMBERS
36'-0 D 392'
8 @ 10'-0" = 80'-0"
FIXED END
BACKWALL OF ABUTMENT
EXPANSION END
BACKWALL OF ABUTMENT
12 WF 40
28'-0"
56'-0"
STEEL FRAMING PLAN - 4-LANE
HIGHWAY BRIDGE
SYMMETRICAL ABT ℄
26'-
25' STAY
24'-0"
No 5 @ 10"
No 4 @ 8"
TYPICAL
WEB 11/16"
TOP FLANGE 1"
BOTTOM FLANGE 1 7/8"
BAR 7" x 1 3/4" x 1'-3 7/8"
6WF15.5
BNo. 5
No 3 TIES @ 12"
ST. 6WF13.5 x 0'-8" @ 5'-0" O.C
No 5 @ SALT BARS BENT
STIFF R.
TAB FLANGE
PLAN OF TOP FLANGE
SECTION A-A

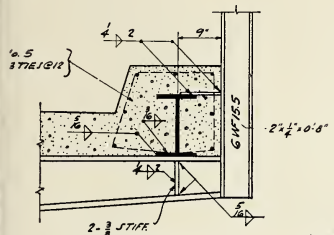


DETAIL "X"

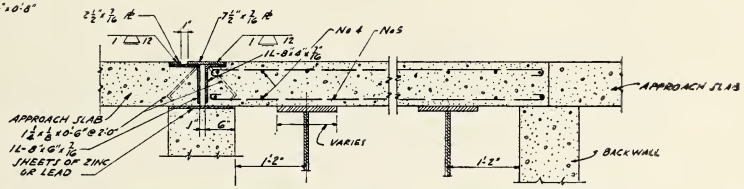


SECTION 3'-3'

SECTION 3-3

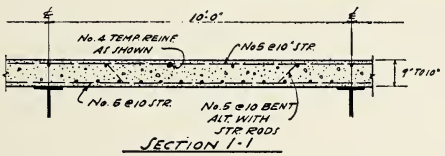


SECTION 2-2

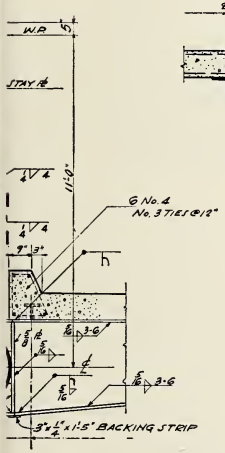


DETAIL OF EXPANSION JOINT

DETAIL OF FIXED END



SECTION 1-1



1-A

DESIGN DATA

LOADING :---------- H15 - S12 - 44
STEEL :---------- A.S.T.M. - A7 - 46
SPECIFICATIONS :---------- A.A.S.H.O. STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES & A.W.J. STANDARD SPECIFICATIONS FOR WELDED HIGHWAY & RAILWAY BRIDGES.

Figure 55

CONTINUOUS TRUSSES

Whereas the number of entrants who used continuous girders for their bridges greatly exceeds those who used simple span girders, the reverse is true for the trussed bridges.

Nan-sze Sih, Whitestone, New York, submitted a continuous deck truss with three spans of 90 ft., 180 ft., and 90 ft. respectively. The entire length of 360 ft. is divided into twenty-four panels each 15 ft. in length. The bridge has three lanes with a total roadway width of 36 ft. The trusses are 28 ft. apart. The depth of the trusses varies. At the piers, the depth is 21 ft., and at the abutments and at the center of the middle span, the depth is 10 ft.

A 7 in. reinforced concrete slab with a 2½ in. crown is designed as a two-way slab. It is supported on the floorbeams which are continuous over three supports and on three lines of continuous stringers. The top chords serve as the two outside lines of stringers. The other stringer is a continuous 16 WF 45 located at the centerline of roadway. The floorbeams receive support at the truss top chords, also at the centerline of roadway from the diagonals of the cross frames as shown in Figure 56.

Mr. Sih made these comments about his design:

"Wide flange or built-up H sections with approximately equal radii of gyration in both directions, are used for the welded bridge which is more economical than the riveted built-up sections; whereas if only wide flange sections are used for the riveted bridge, it will result in approximately the same cost but will require more steel.

"Gusset plates are also used for the welded bridge in order to simplify the end detail of the members, to reduce the length of the welds and to have all main connections butt welded.

"It is assumed that the erection of the welded bridge will be carried out as follows: The bridge may be welded in the shop or at the site in a jig in five sections as shown in Figures 57 and 58 [see field splices]. One falsework will be required to erect the first two end sections and the intermediate sections; the center section will be suspended from the cantilevered intermediate sections by erection bolts before welding. This same method may be applied to the erection of fascia and railings. While this erection procedure will reduce the field weld to a minimum, the design is still adaptable should the number of field splices be increased for other considerations.

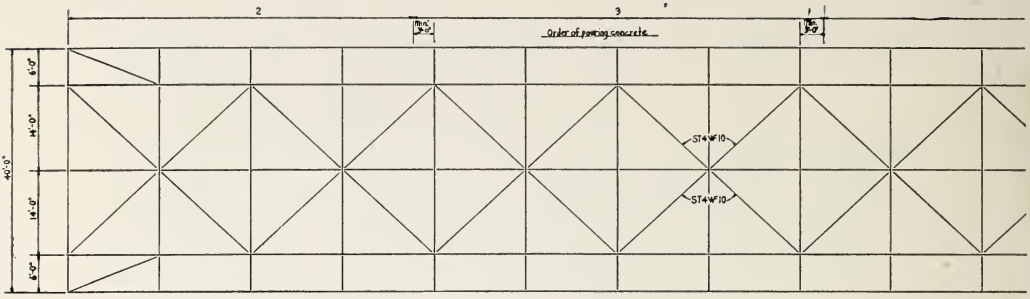
"As an approximate comparison the following estimate is made on the cost of structural steel only, as all other costs are the same.

Riveted Bridge:	264.4 tons @ $400		$105,800
Welded Bridge:			
($6¢$/lb. steel, $5¢$/lb. fabrication, $5¢$/lb. erection and assembly)	187.7 tons @ $320	60,500	
Shop weld 2,616 lb. @ $6		15,700	
Field weld 748 lb. @ $12		9,000	
			$ 85,200

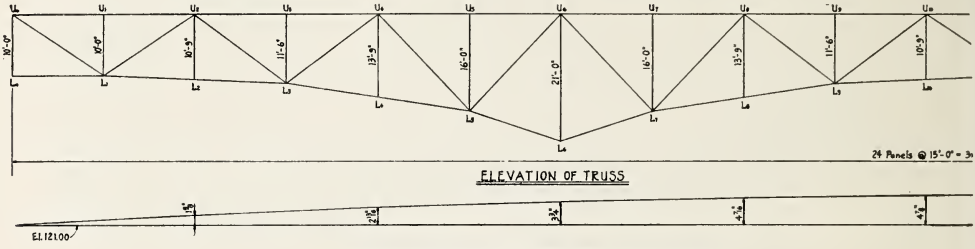
"Conclusion:

1. The welded bridge achieves greater saving on the weight of steel in comparison to riveted bridge and with proper erection scheme and details, cost of construction can also be reduced.
2. Repair and maintenance for the welded bridge are simpler and less expensive than on a riveted bridge on account of its simplicity in detail and in construction.
3. The welded bridge gives a neater and more pleasant appearance."

The details of the riveted bridge are shown in Figures 59 and 60.

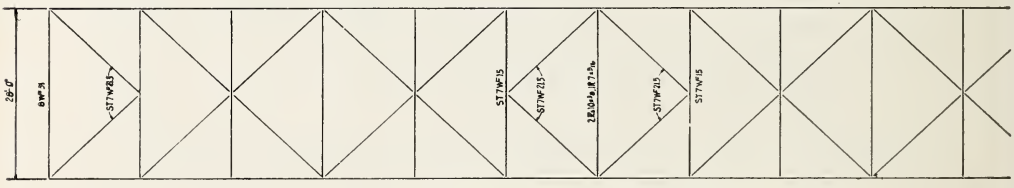


PLAN OF FLOOR AND TOP LATERAL SYSTEM



ELEVATION OF TRUSS

NORMAL POSITION OF TOP CHORD UNDER FULL DEAD LOAD



PLAN OF BOTTOM LATERAL SYSTEM

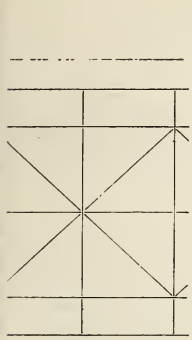
Note: All top and bottom lateral bracings are ST4V
All bottom lateral struts are ST6WF135 exce

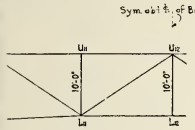
MEMBER	LENGTH	STRESSES DL.	LL.	I.	T_o	E.	WIND	DESIGN STRESS	Allow. STRESS	AREA Reqd.	SECTION	1/r	AREA
L_0L_1	15.00	0	0	0	0	0	——	——	——	——	$2R_8 14 \times \frac{5}{8}$ IR $9 \times \frac{5}{16}$	51	13.88
L_0L_2	30.06	+5	+132	+31	±12	±27	±4	+237	180	132	$2R_8 14 \times \frac{7}{16}$ IR $9 \times \frac{5}{16}$	51	15.63
L_2L_3	30.38	-163	-126	-21	±19	±43	±8	-353	167	243	$2R_8 16 \times \frac{5}{8}$ IR $9 \times \frac{1}{2}$	43	24.90
L_3L_4	15.81	-343	-161	-29	±20	±44	±13	-577	146	355	$2R_8 16 \times \frac{11}{16}$ IR $9 \times \frac{5}{8}$	41	39.63
L_4L_5	15.81	-343	-161	-29	±20	±44	±7	-577	146	395	$2R_8 16 \times \frac{11}{16}$ IR $9 \times \frac{5}{8}$	41	38.63
L_5L_6	30.38	-75	-90	-20	±29	±47	±4	-232	147	161	$2R_8 14 \times \frac{5}{16}$ IR $9 \times \frac{5}{16}$	50	16.19
L_6L_7	30.06	+228	+264	+44	±37	±35	±22	+575	180	320	$2R_8 14 \times 1$ IR $9 \times \frac{1}{2}$	48	32.50
L_7L_8	15.00	+361	+339	+56	±40	±19	±46	+775	180	430	$2R_8 14 \times \frac{11}{16}$ IR $9 \times \frac{5}{8}$	48	43.50
U_0L_1	18.03	+38	+130	+30	±8	±17	——	+215	180	119	10 WF 45	108	13.24
L_1U_2	18.03	+33	+78	+12	±7	±16	——	+139	180	77	10 WF 33	112	9.71
U_2L_2	18.90	-30	-74	-21	±7	±15	——	-200	130	154	10 WF 54	88	15.88
L_2U_3	18.90	+115	+97	+27	±2	±6	——	+245	180	136	10 WF 49	89	14.40
U_3L_3	21.93	-148	-116	-29	±2	±5	——	-298	137	217	$2R_8 16 \times \frac{5}{8}$ IR $9 \times \frac{1}{2}$	73	22.00
L_3U_4	21.93	+89	+101	+24	±3	±6	——	+220	180	122	$2R_8 16 \times \frac{5}{8}$ IR $9 \times \frac{5}{8}$	74	15.43
U_4L_3	21.93	+144	+114	+19	±9	±9	——	+286	180	159	$2R_8 16 \times \frac{5}{8}$ IR $9 \times \frac{7}{16}$	73	16.19
L_3U_5	21.93	-222	-135	-22	±6	±1	——	-380	137	277	$2R_8 16 \times \frac{11}{16}$ IR $9 \times \frac{5}{16}$	72	27.81
U_5L_4	18.90	+209	+141	+23	±7	±1	——	+368	180	205	$2R_8 14 \times \frac{5}{8}$ IR $9 \times \frac{5}{16}$	73	20.88
L_4U_5	18.90	-177	-134	-22	±3	±14	——	-367	140	268	$2R_8 16 \times \frac{5}{8}$ IR $9 \times \frac{7}{16}$	62	24.94
U_5L_5	18.03	+116	+125	+25	±3	±14	——	+280	180	156	10 WF 54	85	15.88
L_5U_6	18.03	-35	-102	-22	±0	±16	——	-173	132	131	10 WF 49	85	14.40
U_6L_6	21.00	-205	-107	-13	±4	±9	——	-334	138	241	$2R_8 16 \times \frac{11}{16}$ IR $9 \times \frac{5}{16}$	70	24.91
U_6L_7	16.00	-18	-36	-11	——	——	——	-65	125	52	10 WF 33	99	9.71
All Vert.	——	——	——	——	——	——	——	——	120	——	10 WF 21	105	6.13

MEMBER	LENGTH	STRESSES DL.	LL.	I	T_o	E	WIND	DESIGN STRESS	MOMENT no. DL.	MOMENT LL+I	Allow. STRESS
U_1L_1	30.00	-32	-98	-26	±7	±15	±9	-138	130	460	18,15
U_1L_2	"	+70	+140	+32	±17	±38	±15	+210	"	660	18
U_1L_2	"	+263	+155	+29	±20	±45	±26	+209	"	"	18
U_1L_2	"	+235	+196	+26	±25	±47	±13	+179	"	"	18
U_1L_3	"	-98	-136	-30	±36	±46	±44	-212	"	460	18,15
U_1L_3	"	-332	-238	-62	±40	±30	±92	-330	"	"	18,15

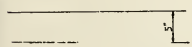
SWAY at Panel	LENGTH	STRESSES DL	LL+I	DESIGN STRESS	Allow. STRESS	SECTION	1/r	AREA
6	25.2	-7	-41	-48	114	$2R_8 10 \times \frac{5}{8}$, IR $7 \times \frac{5}{16}$	119	9.69
7,5	——	-8	-69	-57	114	ST5WF165, IR $8 \times \frac{1}{16}$	120	8.35
all others	——	-10	-59	-69	114	ST4WF12, IR $6 \frac{1}{2} \times \frac{5}{8}$	120	5.97

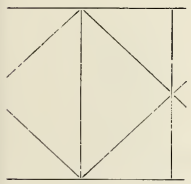
REACTION	DL	LL	I	T_o	E	Trans. WIND	Long. WIND	TRACTION
R_0	41	64	15	±5	±10	±5	——	——
R_8	422	151	37	±5	±18	±39	——	——
H_0	——	——	——	——	——	——	±67	±26





0" = 360'-0"



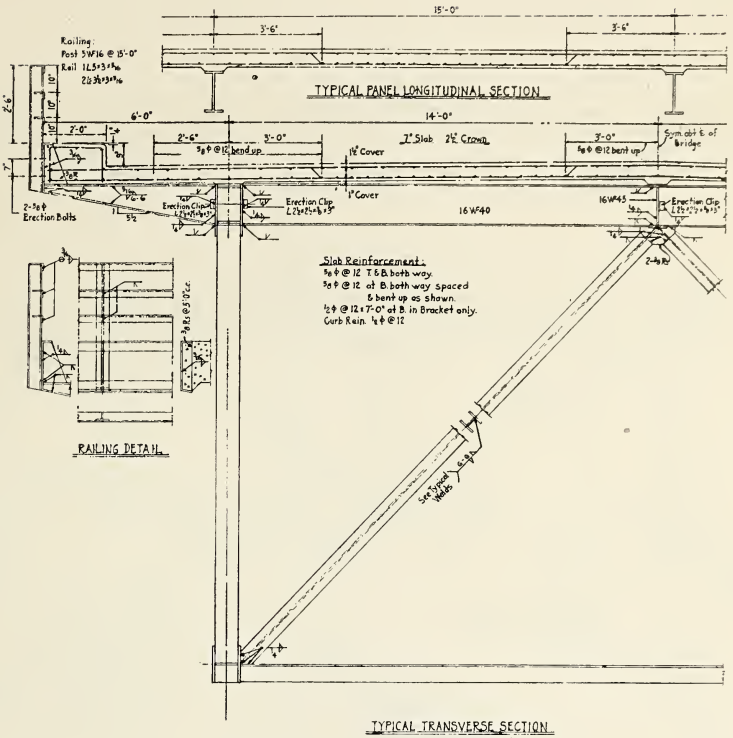


: ST4WF85 except as noted
5 except as noted.

ow. ESS.	SECTION	AREA	SECTION MODULUS	COMPOSITE AREA	COMPOSITE S.M.	UNIT STRESSES AXIAL	UNIT STRESSES BENDING	UNIT STRESSES CONCRETE
16	2B16×7/16 1R9½×5/8	26.13	108	29.76	124	5.97	5.46	.705
	2B16×1/2 " "	26.13	113	27.53	129	10.3	6.0	1.71
	2B16×5/8 " "	30.13	145	39.13	162	12.2	5.4	1.76
	2B16×3/4 " "	34.13	134	35.53	150	11.9	5.8	1.77
15	2B16×1/2 " "	26.13	113	31.73	129	10.4	5.3	.99
15	2B16×15/16 " "	52.13	181	57.73	198	12.1	3.37	9.05

MEMBER	MOMENT D.L.	MOMENT L.L+I	SHEAR D.L.	SHEAR L.L+I	SECTION	AREA	S.M.
Floor Beam	+10	-33	1	35	16WF40	11.77	644
Stringer	-10	±99	6	38	16WF45	13.24	724
Fascia	-13	-16	5	7	2B7×⅜ 2L6×4×⅜	—	—
Bracket	-56	-42	9	21	2B7×7/16 1R15×⅝	13.24	724

T_0 — Stress due to 10°F Differential Temperature.
E — " " " ½° Error of Agjustment



Note:
Design Specifications AASHO 1949 & AWS 1947.
Loading 3 lanes H20S16-44
All Materials Carbon Steel, except as noted.

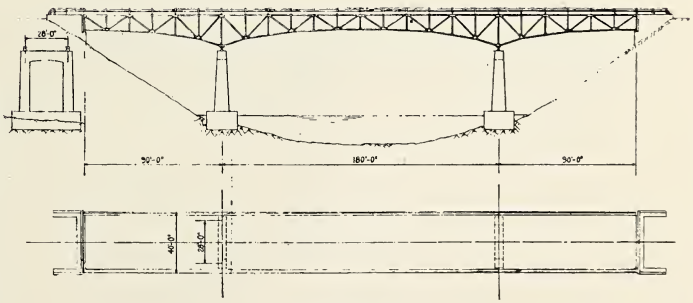
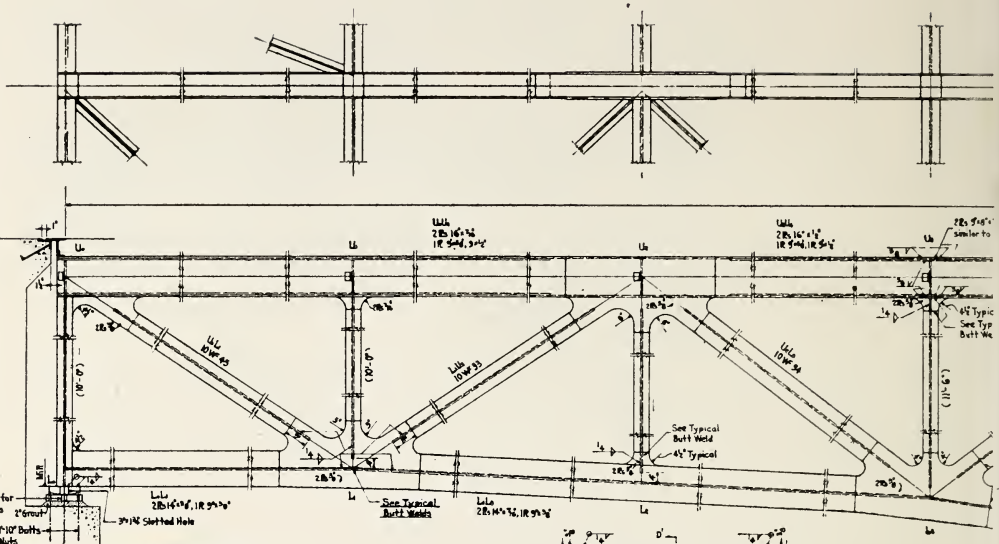
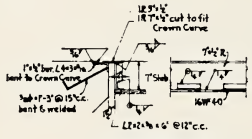
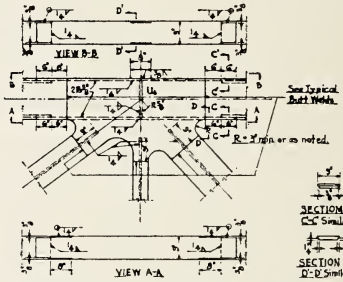


Figure 56



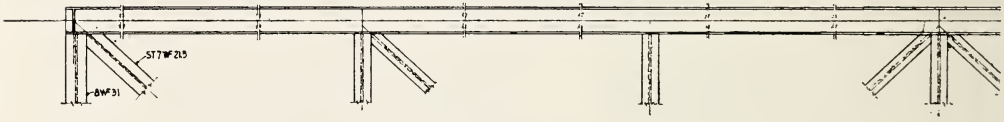


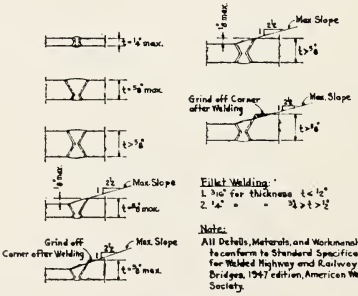
DETAIL FLOOR JOINT
Scale 3/4"=1'-0"



TYPICAL DETAIL TOP CHORD JOINT

Note:
All Joint Details shown are typical.




Max. Slope
Grind off Corner after Welding
Fillet Welding:
1. 5/16" for thickness t < ½"
2. ¼" " " ½" > t > ¾"
Note:
All Details, Materials, and Workmanship to conform to Standard Specifications for Welded Highway and Railway Bridges, 1947 edition, American Welding Society.

TYPICAL WELDS

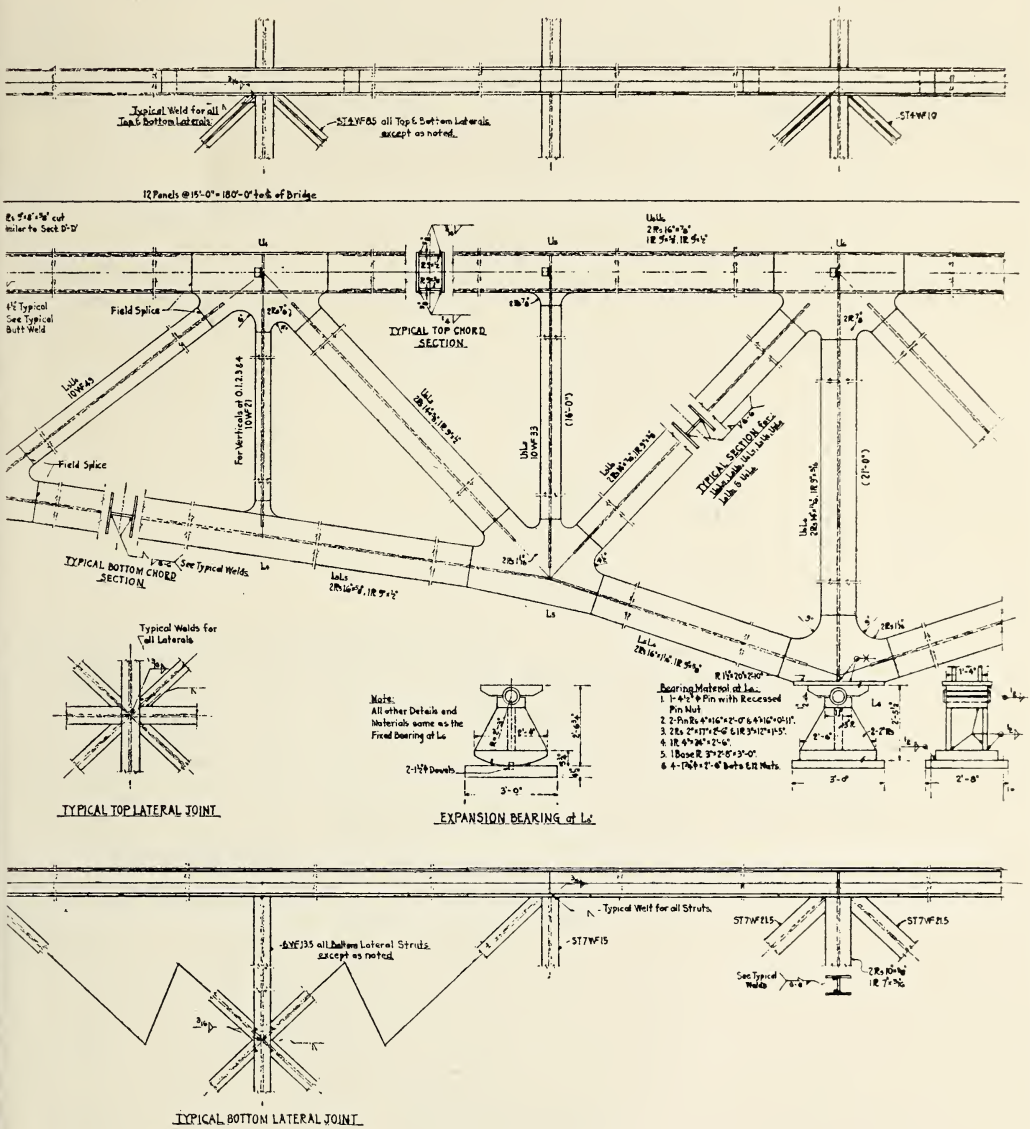
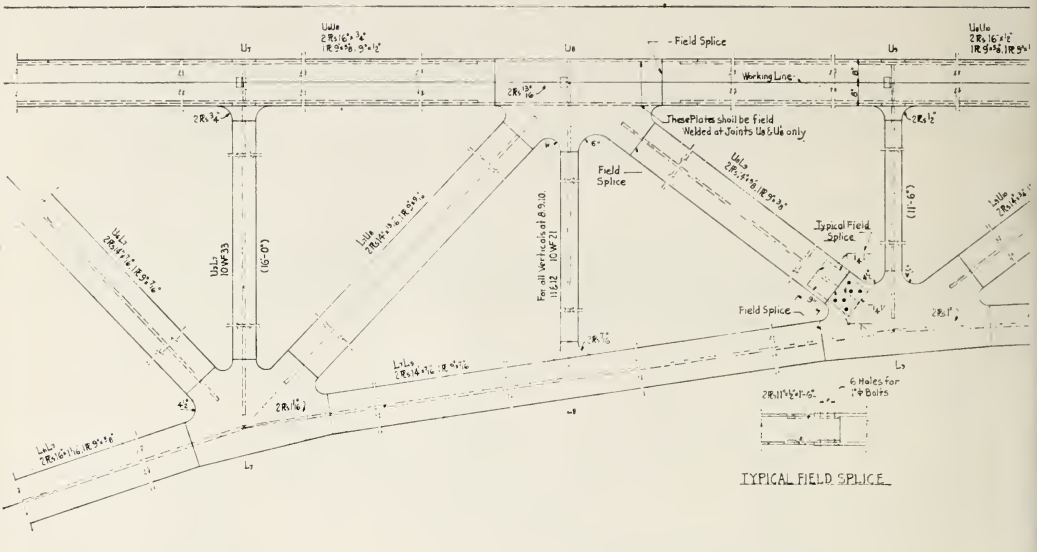

Typical Weld for all Top & Bottom Laterals
ST4WF85 all Top & Bottom Laterals except as noted
ST4WF10
12 Panels @ 15'-0" = 180'-0" back of Bridge
Field Splice
TYPICAL TOP CHORD SECTION
TYPICAL SECTION Ecc.
Field Splice
TYPICAL BOTTOM CHORD SECTION
Typical Welds for all Laterals
TYPICAL TOP LATERAL JOINT
Notes:
All other Details and Materials same as the Fixed Bearing at Lo
2-1½ø Dowels
EXPANSION BEARING at Lo
Bearing Material at Lo
1 1-6½ø Pin with Recessed Pin Nut
2 2-Pin Rs 4"x16"x2'-0" & 4"x16"x0'-11"
3 2 Rs 2"x17"x2'-6" & 1 R 3"x12"x1'-5"
4 1 R 4"x26"x2'-6"
5 1 Base R 3"x2'-8"x3'-0"
6 4-1¼ø x 2'-6" Bolts & 8 Nuts
TYPICAL BOTTOM LATERAL JOINT
Typical Weld for all Struts
ST7WF135 all Bottom Lateral Struts except as noted
ST7WF15
ST7WF215
See Typical Welds

Figure 57



TYPICAL FIELD SPLICE

Deck			
Loc	Mat.	Weight #/Bridge	Total Deck Wt #/Sq
Tran.	$\frac{5}{8}\phi$	100	
Reinf	$\frac{1}{2}\phi$	16	
Long	$\frac{3}{8}\phi$	86	
Reinf	$\frac{1}{2}\phi$	21	
	Total	223	
Slab	7" Conc.	3740	
Welded	Flr Steel	276	4462
Riveted	"	323	4509

Floor Steel					
	Welded		Riveted		
Item	Mat.	Wt #/Sq	Mat.	Wt #/Sq
Stringer	16WF45	43	18WF50	50
Floor Bm	16WF40	73	16WF50	93
Bracket	$\frac{5}{8}\times\frac{3}{8}$	29	16WF50	36
Fascia	$\angle 12\times 3\times\frac{3}{8}$	306	$\angle 3\frac{1}{2}\times 2\times\frac{3}{8}$	18.2
	$2\frac{3}{4}\times\frac{1}{2}\times\frac{3}{8}$	392	$2\frac{3}{4}\times\frac{1}{2}\times\frac{3}{8}$	16.4
Railing	$14.5\times 3\frac{1}{2}\times\frac{5}{16}$	16.4		16.4
	$2\frac{3}{4}\times 3\frac{1}{2}\times\frac{5}{16}$	26.4		26.4
Post	5WF16	8.6		8.6
	$12\times 2\frac{1}{2}\times\frac{3}{8}$	74	Conn Rib	32.9
Total		2756		322.9

Structural Steel (per $\frac{1}{2}$ Truss) Welded Bridge					
Item	Material		Total Weight in lbs.	Item	Mat.	Total Weight in lbs.
Structural Rs Tot'l Weight = 40941 lbs	$15\frac{9}{16}$"	$14'$	1550	Connection Rs	$15\frac{9}{16}$"	3800
	"	$16'$	3540		$1\frac{1}{16}$"	2924
	$1\frac{1}{16}$"	$16'$	2890		1"	1274
	1"	$14'$	2360		$\frac{7}{8}$"	1770
	$\frac{7}{8}$"	$16'$	2450		$13\frac{7}{16}$	932
	$13\frac{7}{16}$	$14'$	1320		$\frac{3}{4}$"	30
	$\frac{3}{4}$"	$9'$	290		$\frac{5}{8}$"	1800
	"	$14'$	995		$1\frac{1}{2}$"	20
	"	$16'$	2100		$\frac{1}{16}$	18
	$\frac{5}{16}$"	$9'$	5080		$\frac{3}{8}$"	780
	"	$14'$	1885		Total	13348
	"	$16'$	1720			
	$\frac{9}{16}$"	$9'$	298			
	$\frac{1}{2}$"	$9'$	3385			
	"	$16'$	2720			
	$\frac{7}{16}$"	$8'$	428			
	"	$9'$	743			
	"	$12'$	3610			
	$\frac{3}{8}$"	$6\frac{2}{3}'$	1220			
	"	$9'$	743			
	"	$10'$	914			
	"	$8'$	436			
	$\frac{5}{16}$"	$7'$	264			
Structural Shapes Tot'l Weight = 4762 lbs	10WF54		454			
	10WF49		1325			
	10WF45		577			
	10WF33		430			
	8WF31		403			
	ST 7WF215		1195			
	ST 7WF15		580			
	ST 6WF135		1320			
	ST 5WF12.5		590			
	ST 4WF10		370			
	ST 4WF8.5		3120			
	ST 4WF12		1760			
	10WF21		1640			

Total Weight of Steel per Bridge:
Rs 40941
W.ST 14764
C Rs 13348
69053 per $\frac{1}{2}$ Truss
1381 Tons/Bridge
Flr Steel 436 " "
Grand Total = 1817 " "
Weight of Truss = 383 lbs./ft truss
" Deck = 4462 lbs/ft Bridge

Weld				
Shop Butt Weld				
Size	Total Length in ft	Unit Wt lbs./ft	Total Wt	
$15\frac{9}{16}$	77	4.78	33	
$1\frac{1}{16}$	107	3.11	32	
1	42	2.71	13	
$\frac{7}{8}$	53	2.2	12	
$13\frac{7}{16}$	47	1.94	9	
$\frac{3}{4}$	107	1.68	18	
$\frac{5}{8}$	208	1.68	35	
$\frac{9}{16}$	47	1.42	9	
$\frac{1}{2}$	136	1.15	16	
$\frac{7}{16}$	255	.92	24	
$\frac{3}{8}$	422	.7	30	
$\frac{5}{16}$	6	.46	3	
		Total	234	
Shop Fillet Weld				
$\frac{1}{4}$	895	28	250	
$\frac{5}{16}$	1136	15	170	
		Total	420	

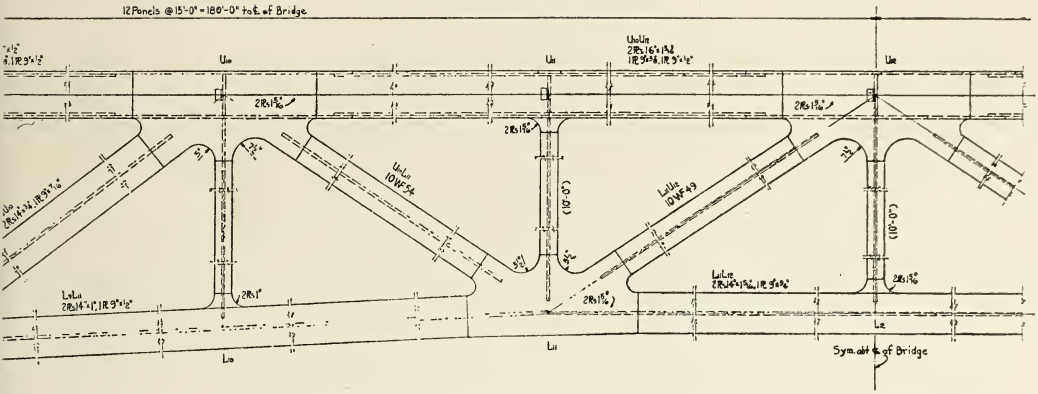
Total Shop Weld = 654
" Field " = 187
841

Total Shop Weld per Brid
" Field " " "

Grand To

Shop Weld lbs/ton of St
Field " " "

QUANTITY E



Welds (per ½ Truss)

Field Butt Weld				
Total Wt lbs.	Size	Total Length in ft.	Unit Wt. lbs./ft	Total Wt lbs
33	3/8	146	168	25
32	1/2	55	115	63
13	7/16	22	92	20
12	3/8	325	.7	23
9	5/16	260	.46	12
18			Total	143
Field Fillet Weld				
35	1/4	255	.28	7
16	3/16	242	.15	37
24			Total	44
30				
3				
234				
250				
170				
420				

654 lbs. per ½ Truss
187 " " "
841
er Bridge = 2616 lbs.
" " = 748 "
ond Total 3364 #/Bridge

of Structural Steel = 14
" " " = 4

Y ESTIMATE

Structural Steel (Per ½ Truss) Riveted

Item	Material	Total Weight in lbs.
Structural Shapes Total Weight = 65440 lbs.	15 I 339	9180
	15 I 10	2400
	15 I 50	19700
	12 WF 106	2120
	12 WF 92	3400
	12 WF 85	1530
	12 WF 79	1500
	12 WF 65	3360
	12 WF 58	1860
	12 WF 50	800
	12 WF 53	800
	12 WF 40	1860
	12 WF 27	2900
	10 WF 49	640
	8 WF 31	810
	8 WF 17	2100
	ST 8 WF 44	370
	ST 8 WF 39	1480
	ST 7 WF 215	4460
	ST 4 WF 10	360
	ST 4 WF 85	2690

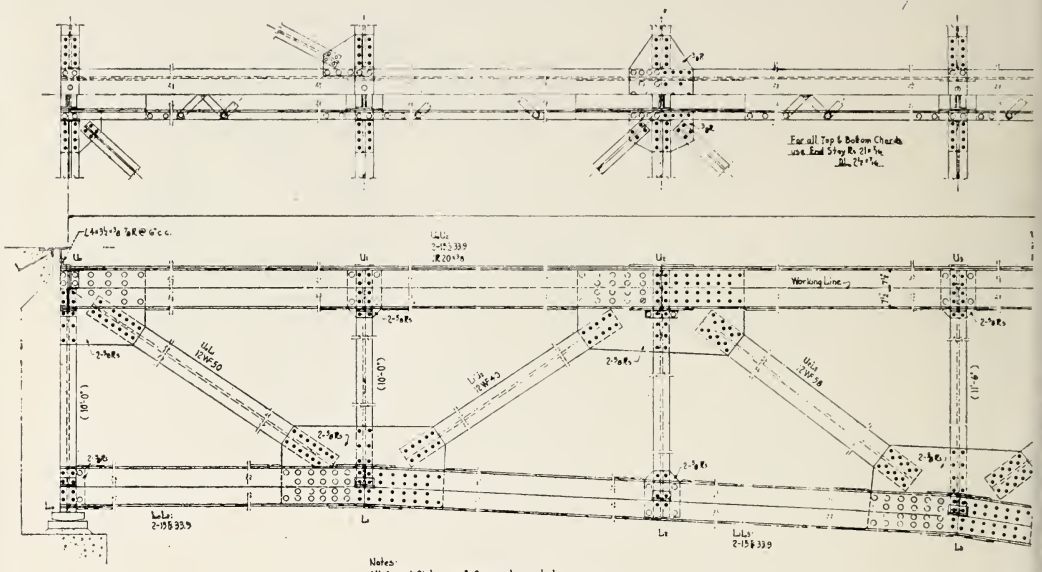
Item	Mat.	Total Weight in lbs
Plates 12800 lbs	22 1/4 x 3/8	3060
	22 x 3/8	3380
	14 x 3/8	2160
	14 x 7/16	1250
	14 x 5/8	1790
	14 x 13/16	1160
Cons. Rs 17140 lbs.	5/8"	12200
	3/8"	3920
	others	1000
5750 lbs	7/8" Stay Rs	2000
	2½" ∠ 2½" ∠ 3/8	3750

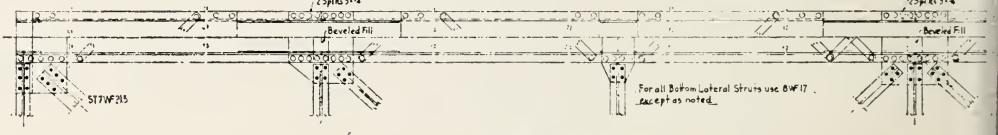
Total Weight of Steel per Bridge:

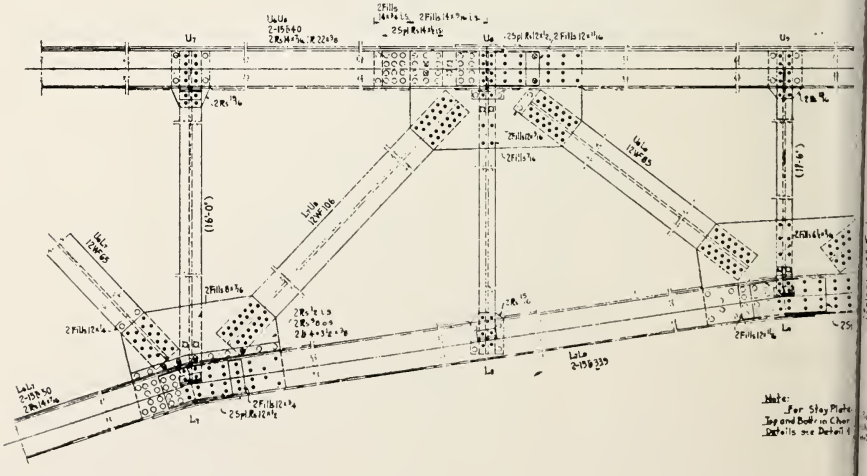
B.W.P.S.I.	65440
Rs	12800
C. Rs	17140
S.Rs, Pl.	5750
	101130 per ½ Truss
(inc. 3% Rivet Heads)	208.3 Tons/Bridge
Fin. Steel	58.1 "
Grand Total =	266.4 " "

Weight of Truss = 562 lbs/ft of Truss
" Deck " = 4543 lbs/ft " Bridge

Figure 58


For all Top & Bottom Chords use End Stay PL 21×¾
Working Line
Notes:
All Gusset Plates are ⅜R, except as noted
All Lateral Plates for Top & Bottom Laterals are ⅜R except as noted.
For Bearings and Bearing Materials see Sh. 2
For End Span Floor Breaks at "U" & "D" see Sh. 2 except as shown
All Rivets are ⅞


Beveled Fill
ST 7WF 215
For all Bottom Lateral Struts use 8WF 17 except as noted


Note:
For Stay Plate Top and Bottom Chor Details see Detail

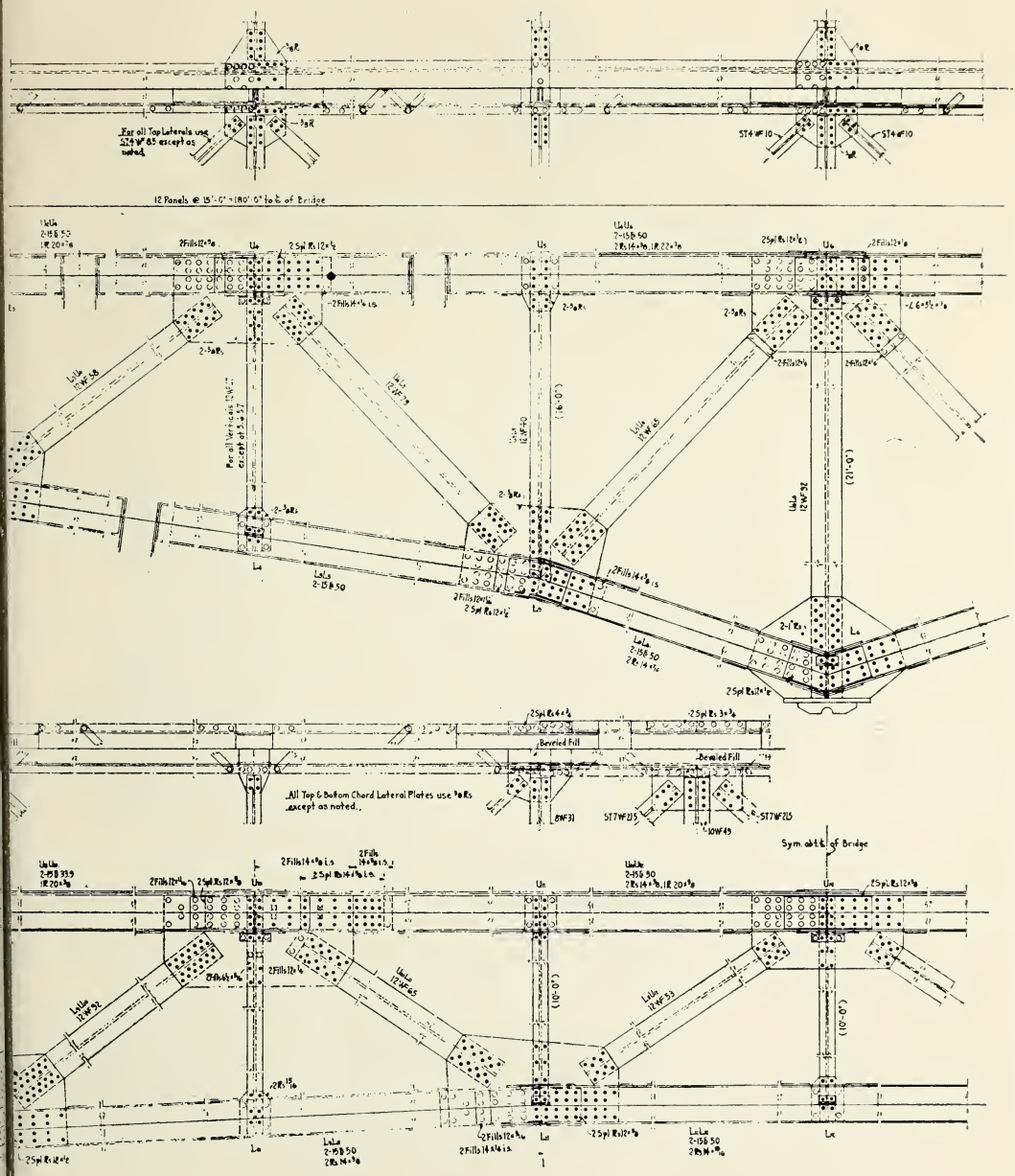

For all Top Laterals use ST4WF85 except as noted.
ST4WF10
12 Panels @ 15'-0" = 180'-0" to ℄ of Bridge
Beveled Fill
All Top & Bottom Chord Lateral Plates use ⅜ Rs except as noted.
Sym. about ℄ of Bridge
Plates and Lacing Bars for Chords and all other similar, Shall for End Span U0 to U2

Figure 59

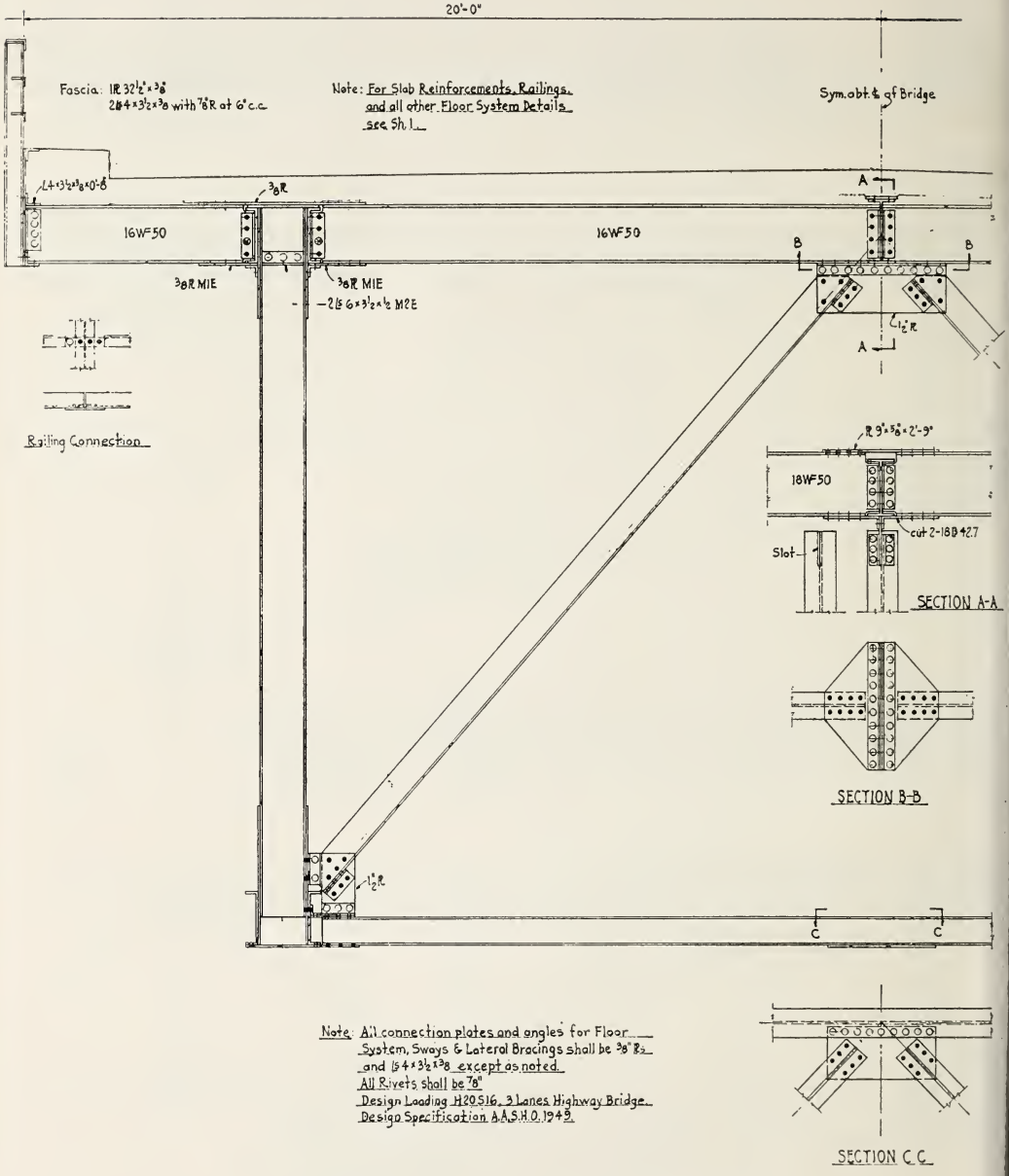

20'-0"
Fascia: 1R 32½"x⅜"
2L4x3½x⅜ with ⅞R at 6" c.c.
Note: For Slab Reinforcements, Railings, and all other Floor System Details see Sh 1.
Sym. about ℄ of Bridge
L4x3½x⅜x0'-8"
⅞R
16WF50
16WF50
⅞R MIE
⅞R MIE
-2PL 6x3½x½ M2E
A
A
B
B
1½"R
Railing Connection
1R 9"x⅝"x2'-9"
18WF50
cut 2-18B42.7
Slot
SECTION A-A
SECTION B-B
1½"R
C
C
Note: All connection plates and angles for Floor System, Sways & Lateral Bracings shall be ⅜" thk and L4x3½x⅜ except as noted.
All Rivets shall be ⅞"
Design Loading H20 S16, 3 Lanes Highway Bridge.
Design Specification A.A.S.H.O. 1949.
SECTION C C
TYPICAL TRANSVERSE SECTION

Figure 60

CHAPTER VI

ARCHES

Slightly more than one-sixth of the designs presented in this program are arch bridges. Some of the arches are trussed—others have solid ribs. There are many differences between the designs. Among these differences are: the manner in which the longitudinal thrust is resisted, by abutment reaction or by tie girders; the members proportioned to resist most of the bending moment, the arches or the stiffening girders; and the location of the roadway deck. One arch bridge is continuous for three spans.

A. R. Werth, Brooklyn, New York, designed a three span bridge using what he termed "self-anchored arches". The spans are 450 ft., 900 ft., and 450 ft. (see Figure 61). Two 24 ft. roadways are separated by a 2 ft. wide median. The deck is an 8 in. reinforced concrete slab designed to act compositely with the main longitudinal structure. The arch is a box, 30 ft. wide, which varies in depth from 7 ft. to 15 ft. as shown in Figure 62. This box is stiffened longitudinally on all four sides.

Mr. Werth presented the following discussion of his design.

"It has been a major aim of the writer to show details herein in a generalized fashion, so as to suggest the adaptability of these details to other spans, design systems and types of construction.

"One of the main features of this entry is the presentation of means whereby welded structures may properly enter into the domain of trussed bridge and, in addition, penetrate well within the province of the suspension type bridge.

"The system developed in this entry shall be tentatively called self-anchored arches. This term embraces in great degree the basic concept of the action and, in point of fact, has already been used in connection with suspension bridges without exterior anchorage. More strictly, however, the system is a combination of girders and arches, inasmuch as the end and center portions act as girders, i.e. no axial force exists.

"In addition to this system, the writer has provided an alternate, shown on Figure 63. The alternate is a structure with a tie which, extending from one end of the three-span structure to the other, has no intermediate connection with the arches; i.e. the arches are anchored at the ends only. A genuine self-anchored arch type of structure, this structure is in actuality an inverted self-anchored suspension bridge. It may be noted parenthetically that in order

that this system achieve its optimum expression, the tie ought be a cable. Thus the most favorable internal stress condition is obtained, namely, compression in the welded structure, while tensile forces are exclusively confined to the cables. Such an arrangement results, no doubt, in extreme economy.

"It is easy to envisage that from the presented conceptions and details many other arch types, especially top-tied arch type systems can be derived. It shall be mentioned that several arches can be advantageously tied together; the architectural effect of such a structure is supreme. (In order to obtain a sufficient stiffness of such a structure more than one of the arch bearings should be fixed).

"There is another conception of the presented system: it is a continuous girder with the web cut out in the vicinity of the intermediate supports. In such a view it is a girder with a very effective varying moment of inertia. (Compare influence lines on Figure 63 and notice that the maximum moments at the center are only about 8 per cent of those over the supports).

"In regard to the composition of the main members of the structure, a broad application of box units is made. The principal box units are the girder boxes and the arch boxes. Large single boxes without intermediate webs, they follow the principle that material is most economically used when it is the greatest possible distance from the centroid of the section. This approach was possible because the shears in this type of structure are of an insignificant order and because the reactions and tie forces are introduced and distributed over the entire box width through transverse members of great torsional stiffness.

"The box principle is employed throughout the whole structure, from the biggest to the smallest units. The question of interior corrosion of inaccessible small boxes is herewith mentioned only in passing, since the writer shares the prevalent view that the possibility of corrosion is remote, inasmuch as the boxes are effectively sealed.

"One of the major features in the application of the box principle is the use of half-pipes as stiffeners for the main box walls. The half-pipes are welded to the box walls and, acting in concert with the walls, they make for units of high torsional and buckling stiffness. The stiffness action of such stiffeners is much greater than that of single-web stiffeners. Moreover, a striking economy is afforded. This is readily seen from the fact that the half-pipe furnishes two stiffeners, yet requires half of the amount of welding for two single-web stiffeners.

"Welded big box girders are manifestly the ideal technical and economical solution of great and even medium sized bridges. It is the belief of the writer that the box girder of the submitted design comprises an important contribution to the development of big box girder structures, whose chief problem is the provision of adequate and economical stiffening of large surfaces.

"There is no necessity to emphasize the advantages of welded box structures. It suffices to say, that the box structure is a monolith in the broadest sense of the word—a bridge of one cast—with all its attendant technical and economical advantages.

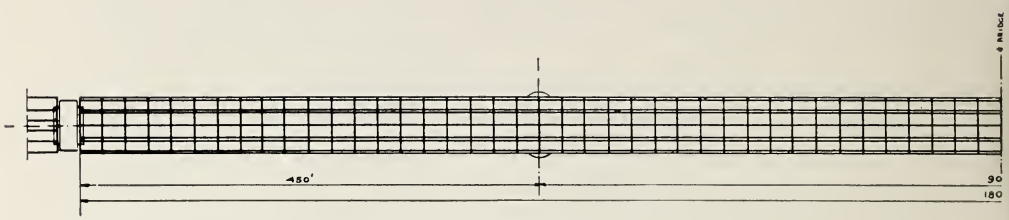
"The details of this entry are applicable to all types of box structures. As to these details a few notes shall help to explain them:

1. The shape of the arch is an empirical one found by several trial designs. In the elevation boxes have to be shaped polygonally for stability reasons. The writer thinks that there is no objection to this from an architectural point of view. Besides, there is a simple way of breaking the corners by adding a curved vertical shield to each of the four edges, which would consist of a plate about 6 x ⅜ in.
2. In the welded structure transverse stiffness is achieved by rigid frames and by cross bracing in the riveted structure. In the welded structure heavy cross bracing is added at the ends of the girders and at the tie connections only.
3. The writer considers that a box of about 2 ft. by 3 ft. minimum dimensions is fully accessible if properly provided with manholes.

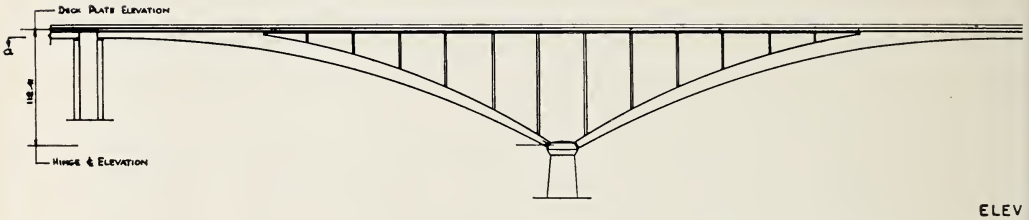
"The roadway design features a composite slab with a concrete deck poured atop a steel plate. There are many structural ways of achieving such a composite slab. The one selected consists of bar trusses which are welded to the steel plate by means of plugs composed of heavy flat steel bent around the web bars (see Figure 64). These plugs are welded at the bottom to the plate and at the top to the chord bar.

"The bar trusses provide support for the steel plate during the pouring of concrete. With the setting of the concrete, the trusses act as shear connectors for the composite slab. The design of the plugs originated with tests made by the writer, which indicated that a direct connection between bent bars and plates failed relatively quickly because of stress concentration at the ends of the bars. Moreover, the direct connection offered insufficient shear resistance.

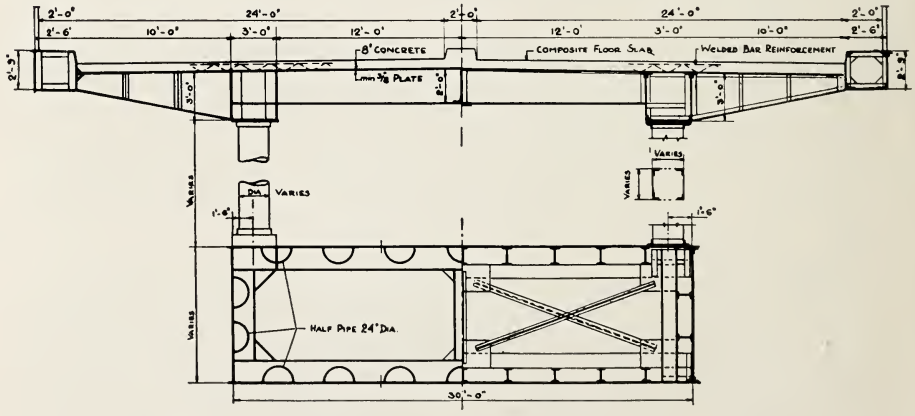
"The advantages of the composite slab as presented may be out-



TOP VIEW
(DECK REMOVED)

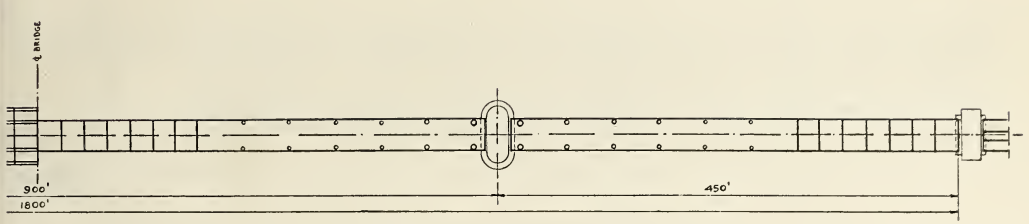


ELEV

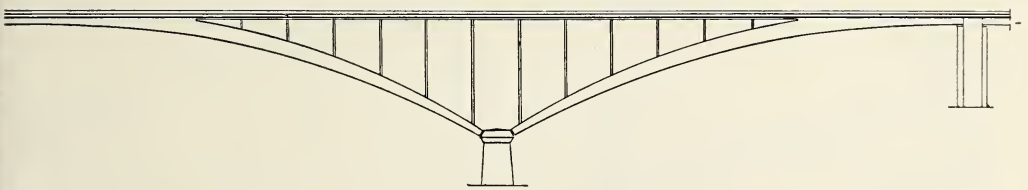


WELDED RIVETED

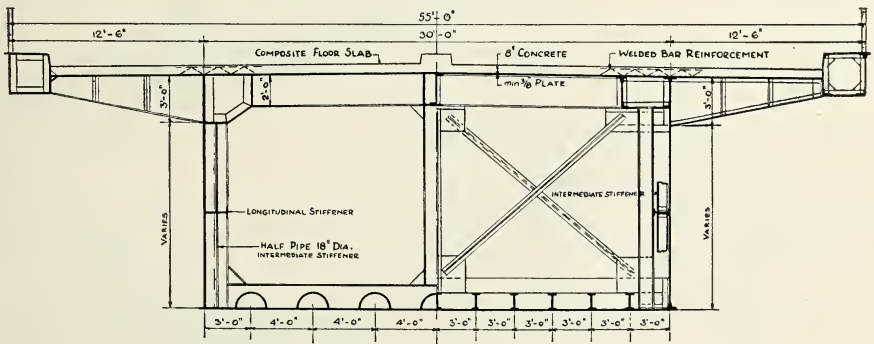
TYPICAL ARCH CROSS SECTION



ARCH PLAN
HALF SECTION a-a



LEVATION

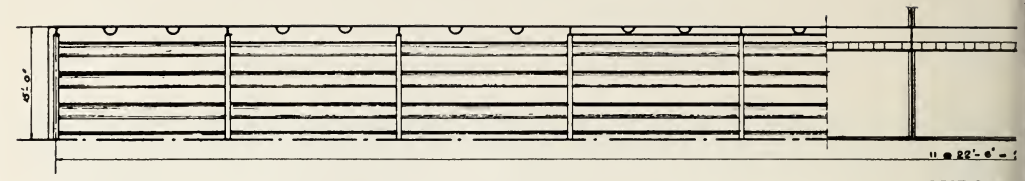


NOTE:
CORRESPONDING DIMENSIONS OF ARCH CROSS SECTION ALSO APPLY TO THIS SECTION AND VICE-VERSA

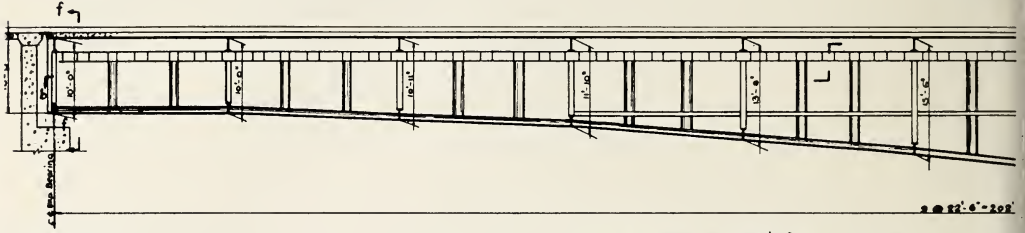
WELDED RIVETED

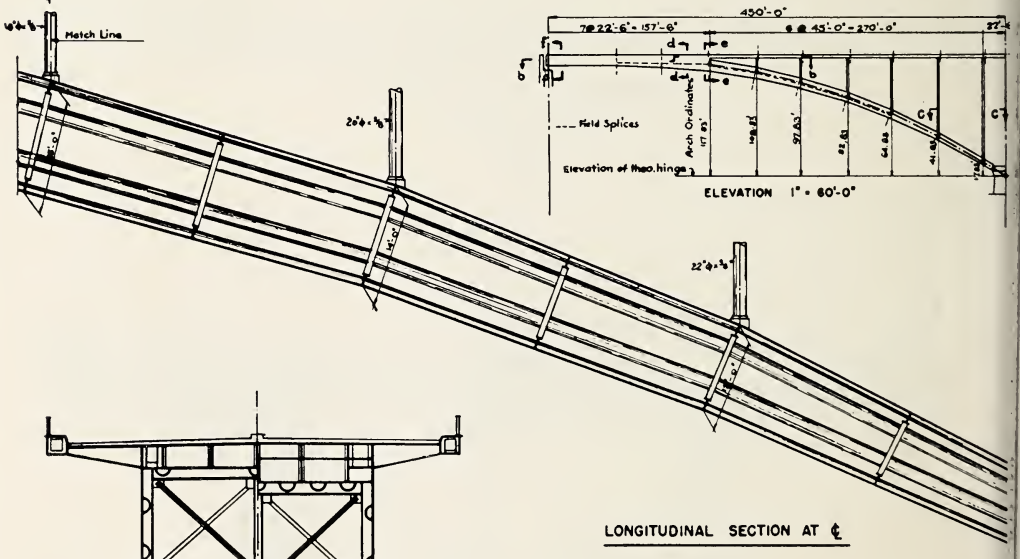
TYPICAL GIRDER CROSS SECTION

Figure 61

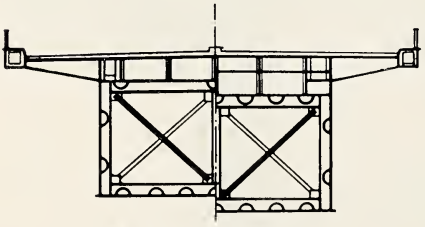


SECTIONAL

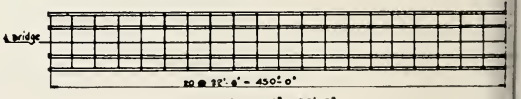




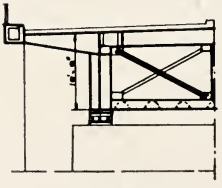
LONGITUDINAL SECTION AT ℄



SECTION d-d SECTION e-e



PLAN 1" = 60'-0"



SECTION f-f

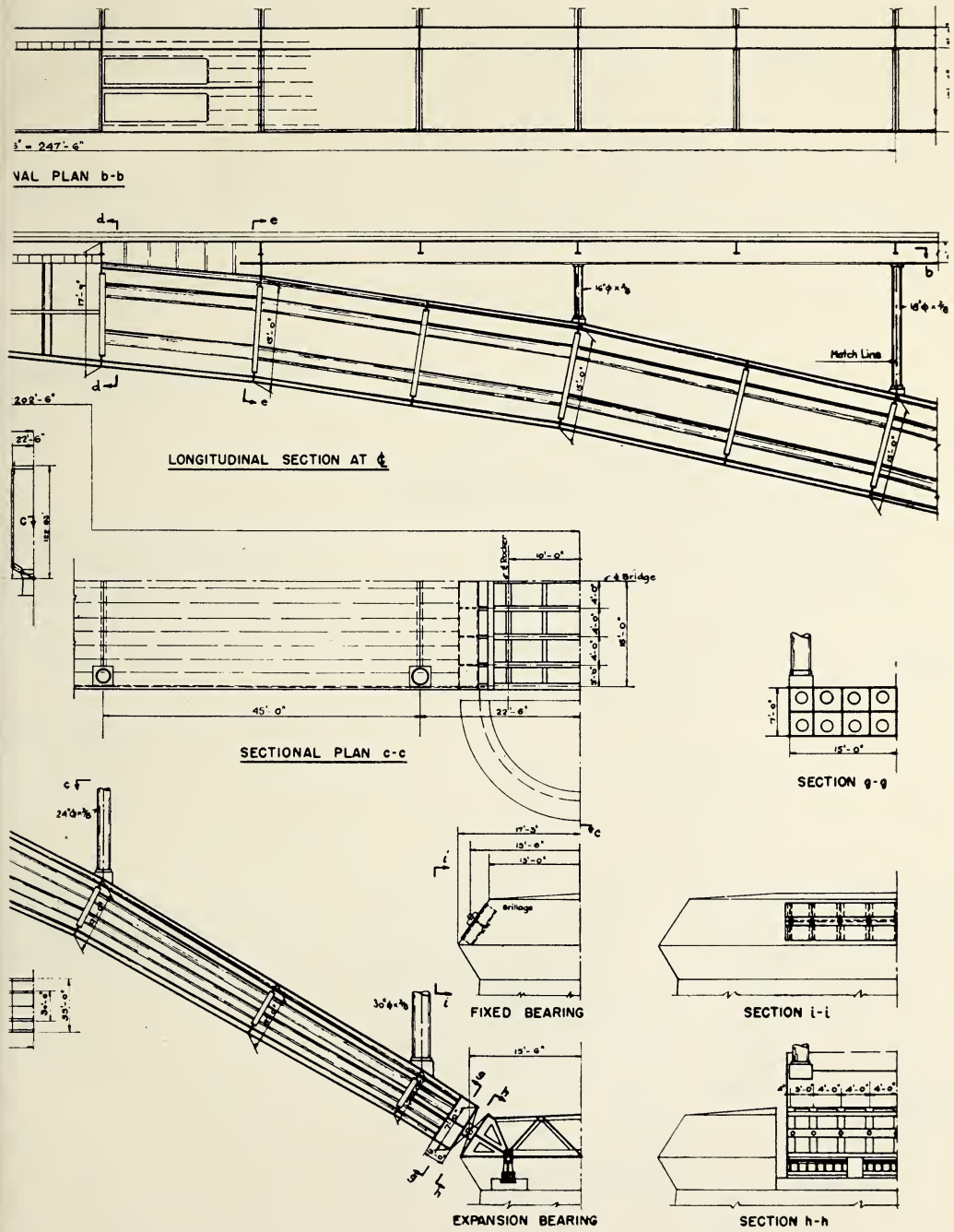
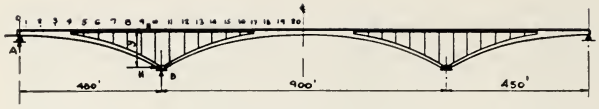
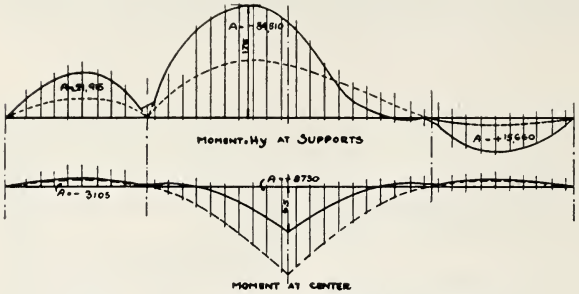
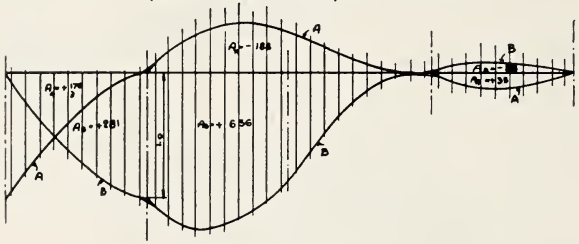

VAL PLAN b-b
LONGITUDINAL SECTION AT ℄
Match Line
SECTIONAL PLAN c-c
SECTION g-g
FIXED BEARING
SECTION i-i
EXPANSION BEARING
SECTION h-h

Figure 62





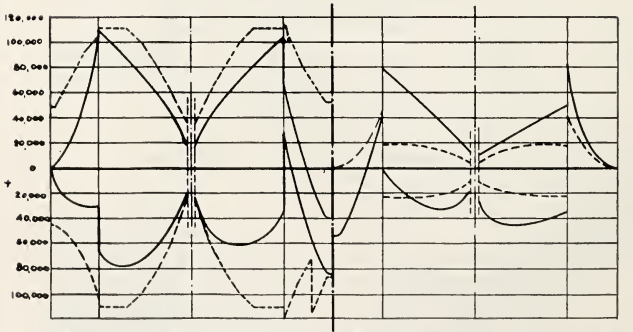
(compare --- Lines of girders with constant moment of Inertia)
(Scale of Ordinates 1" = 100')



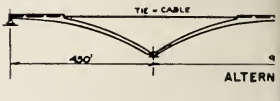
INFLUENCE LINES

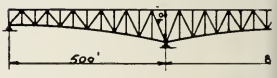
STRESSES

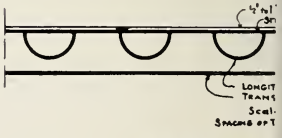
Maximum Tie Stress H	=	1,106,000 k
Maximum Mom. Moment Hy	=	8990 k
Maximum Moment (Pos) at Center	=	+88,200 k
Minimum Reaction A (no uplift)	=	(+) 17 k
Maximum Reaction B	=	10 400 k
Shears insignificant		

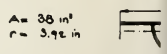


MOMENT DIAGRAMS

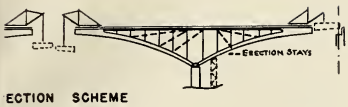




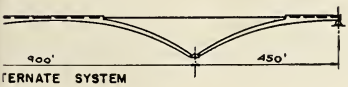




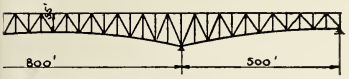
H 20 - S16 - 44, including provision fo
less than 600 feet
For loaded lengths of 600 feet and
600 to 799 ft ~ 640 #/sq in with concen
800 to 999 ft ~ 640 " without c
1000 ft 1199 ft ~ 600 " "
1200 ft and over ~ 560 " "



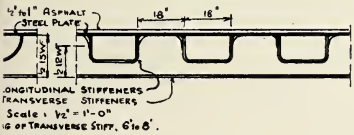
ECTION SCHEME



TERNATE SYSTEM



IVENTIONAL TRUSS



Scale 1 ½" = 1'-0"
IG OF TRANSVERSE STIFF. 6'to 8'.

E STEEL DECK DESIGNS

I = 580 in⁴
L/r = 69 (for L = 22'-6")

TIES OF BOX WALL UNIT
(AS USED IN BRM. B)

LOADS

Dead Load
A.A.S.H.O. 49
Live Load
on for overload in one lane, for loaded lengths of

and greater, live load per lane shall be as follows:
oncentration of 9000# for Moments and 13000# for Shear
ut concentration
"

SUMMARY OF WELDS

TOTAL LENGTH OF WELDED JOINTS			189,000 FEET
	AUTOMATIC AND SEMIAUTOMATIC WELDS	HAND WELDS	SUBTOTAL
IN SHOP	15,700#	4,600#	20,300#
IN FIELD	19,700#	5,800#	25,500#
SUBTOTAL	35,400#	10,400#	
TOTAL GROSS WEIGHT OF WELDS (ELECTRODES)			45,800#
WEIGHT OF WELDS PER TON OF CONSTRUCTION			10.2#

SUMMARY OF WEIGHTS

A. RIVETED STRUCTURE

	LBS ASTM-A7-46	LBS SILICON	LBS TOTAL
1. Deck Plates	2,219,154		
2. Fascia Boxes	968.122		
3. Stringers	212.914		
4. Corner Boxes	1,157.278		
5. Web Plates	385040	940.503	
6. Arch Top Plates	—	848.044	
7. Arch & Girder Bottom Plates	574,479	800.000	
8. Longitudinal Stiffeners	16,705	1,060.000	
9. Floor Beams & Brackets	399,447		
10. Transverse Web Stiffeners	48,864		
11. Cross Frames	399,036		
12 Columns	556.472		
13. Transverse Box at Hinge	150,504		
14.	7,008,013	3,692.547	10,700.612
15. Rivet Heads			299,388
			11,000,000

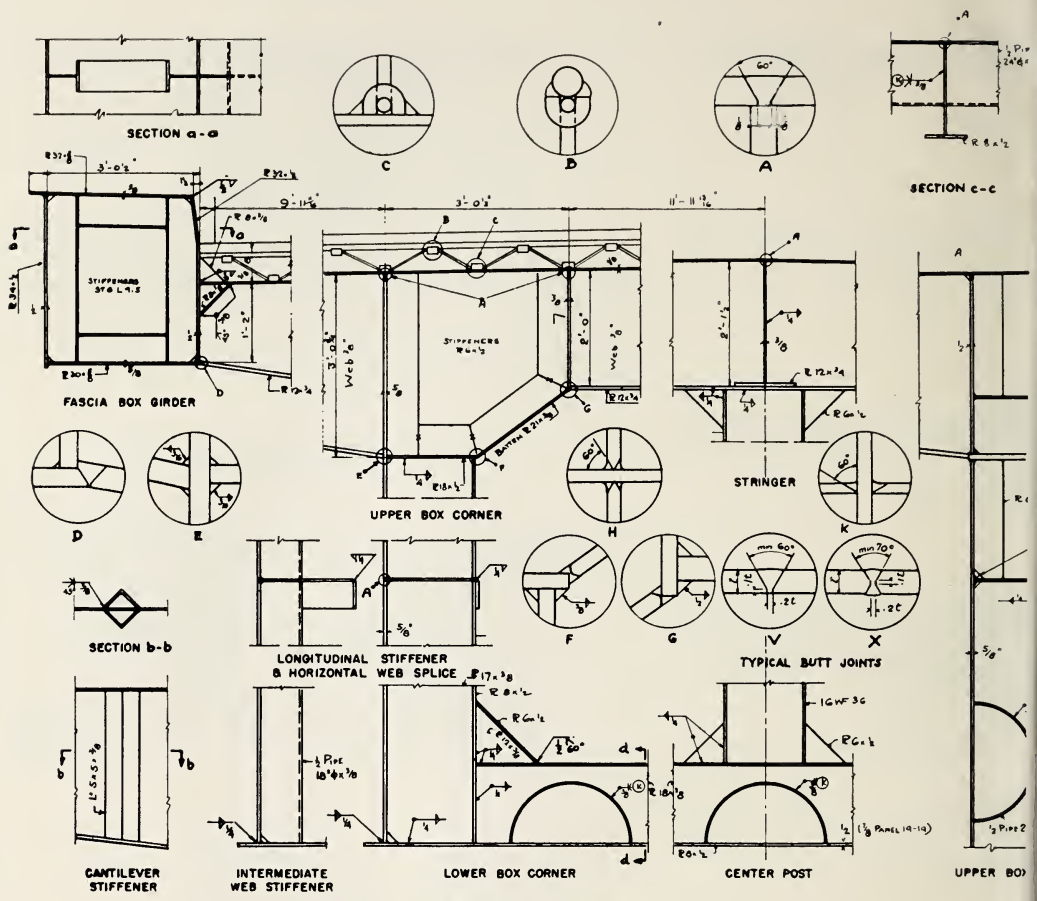
B. WELDED STRUCTURE

	LBS ASTM-A7-46	LBS ASTM A242-46	LBS TOTAL
1. Deck Plates	1,732,725		
2. Fascia Boxes	885,865		
3. Stringers	194,774		
4. Corner Boxes	876,194		
5. Web Plates	272,784	910,148	
6. Arch Top Plates	—	751,470	
7. Arch & Girder Bottom Plates	457,824	737,708	
8. Longitudinal Stiffeners	17,596	1,242,903	
9. Transvers Stiffeners	30,499		
10. Floor Beams & Brackets	258,351		
11. Cross Frames	235,199		
12. Sway Bracings	32,166		
13. Columns	292,909		
14. Transverse Box at Hinge	108,717		
	5,303,603	3,642.229	8,947,832
Welds			45,800
			8,993,632

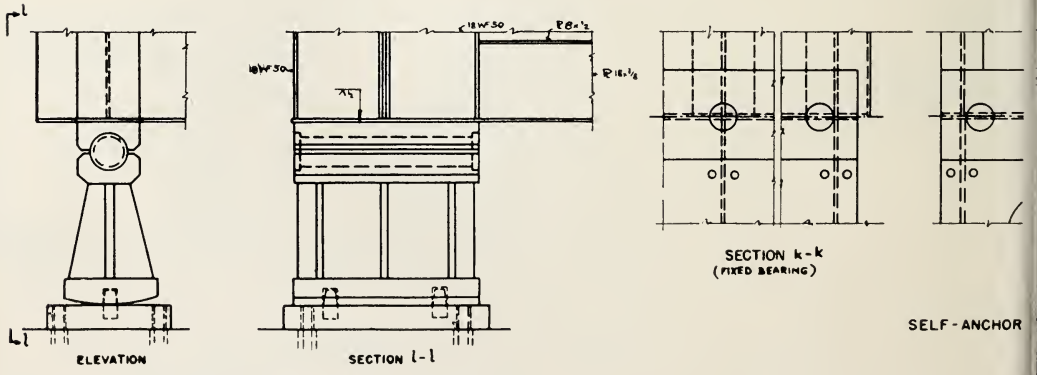
C. CONVENTIONAL TRUSS

	LBS ASTM-A7-46	LBS SILICON	LBS TOTAL
TOTAL	5,340,000	5,488,000	10828,000

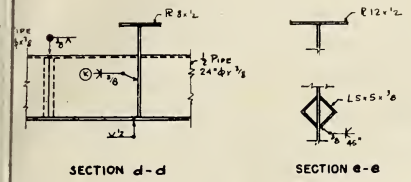
Figure 63

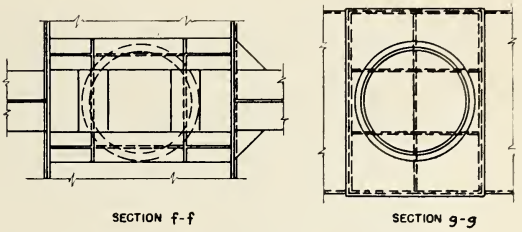


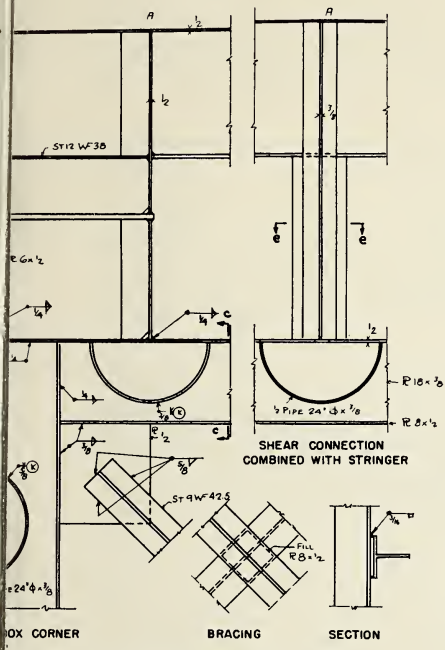
TYPICAL GIRDER SECTION DETAILS

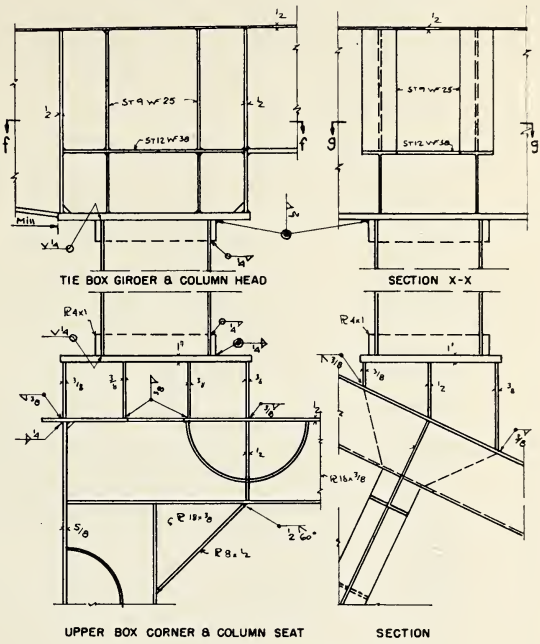


BEARING DETAILS



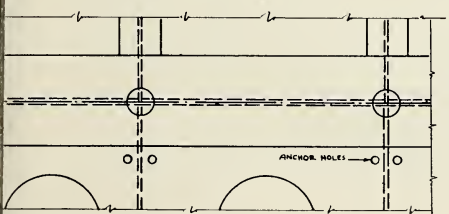






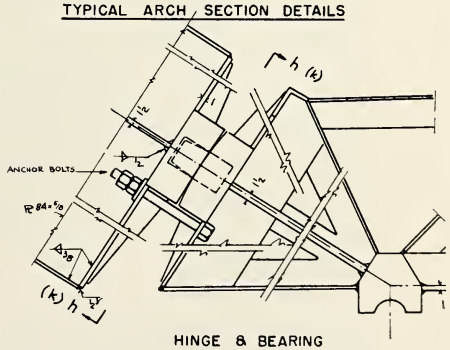
ARCH & GIRDER CONNECTION DETAILS

(For Location See Sheet 2, Section e-e)



ORED ARCHES

TYPICAL ARCH SECTION DETAILS



NOTES

Fillet Welds Not Designated Are 3/16"

Figure 64

lined as follows:

1. With respect to the roadway, the composite slab with a concrete thickness of 8 in. possesses about double the stiffness of a conventionally reinforced slab of 8 in. thickness. (The ratio of stiffness increases sharply with decreasing concrete thicknesses.)

2. With respect to the main structure, the composite slab comprises an economical and effective means of stiffening the top plate of the box.

"In the opinion of the writer, the provision of a concrete deck atop the steel plate, being advantageous both technically and economically, is certain to find wide usage in the future, wherever heavy decks are considered.

"As alternates to this roadway design, lightweight battledeck roadways, as shown on Figure 63, have been studied by the writer and found to be less economical in the present case. However, there is no doubt that such decks, used in conjunction with box stiffeners of semicircular or rectangular section, would prove to be the cheapest and best roadways for structures with larger spans or smaller rises than those of the entry.

"In order to show the economical advantages and the technical implications of the present design, the writer has used all the design data—such as loading, roadway width and pavement weight—of a conventional riveted trussed deck bridge which has been designed in 1951 and 1952 by a prominent bridge-builder and which is now under construction. At the outset it was the writer's intention to submit this design as his Exhibit A. Exhibit B would have served the same purpose as this bridge and the same investment with respect to the substructure was involved for both. However, the type of structure envisioned by the writer differed from the contemplated Exhibit A; moreover, the spans of Exhibit B, the middle one of which was increased for architectural reasons, also differed from the contemplated Exhibit A. Hence the writer developed an original Exhibit A, based, however, upon the same criteria as that of the aforementioned bridge. This background of the choice of exhibits is presented in order to show that Exhibit B can be evaluated in relation to the conventional bridge as well as the writer's own Exhibit A.

"With respect to the conventional bridge, it is to be noted that the writer penalized himself to an extent by increasing the middle span. In addition, the writer had to introduce a counterweight at the junction of the arch and the girder sections of the exterior spans in order to balance the arch moments and to prevent uplift

at the end bearing.

"The comparison of the weights is given in summaries on Figure 63 which are based on enclosed estimates [omitted here]. In the estimates no deductions have been made for scrap such as for perforations, manholes and other, since it was considered, from the point of view of steel conservation, that this material is fully wasted.

"It is shown that the welded bridge is the most economical not only inasmuch as material is concerned; there is a great probability that also the cost of Exhibit B might be less than those of Exhibit A and the conventional truss.

Table of Weights and Costs

Type of Bridge	Steel Weight in % Carbon Steel	Alloy & Silicon respect.	Total (Exh. B 100%	Weight in 1000 lbs.	Unit Price	Total Cost
Exhibit A	79%	43%	122%	11,000	.24	2,640,000
Exhibit B	59%	41%	100%	8,994	.27	2,428,380
Conventional Truss Bridge	60%	61%	121%	10,828	.23	2,490,440

"By refining the design of Exhibit B—a refinement which can be achieved only with the execution of the final design for construction —a saving in weight of about 5 per cent can be safely predicted.

"For the comparison of costs, the following assumptions have been made:

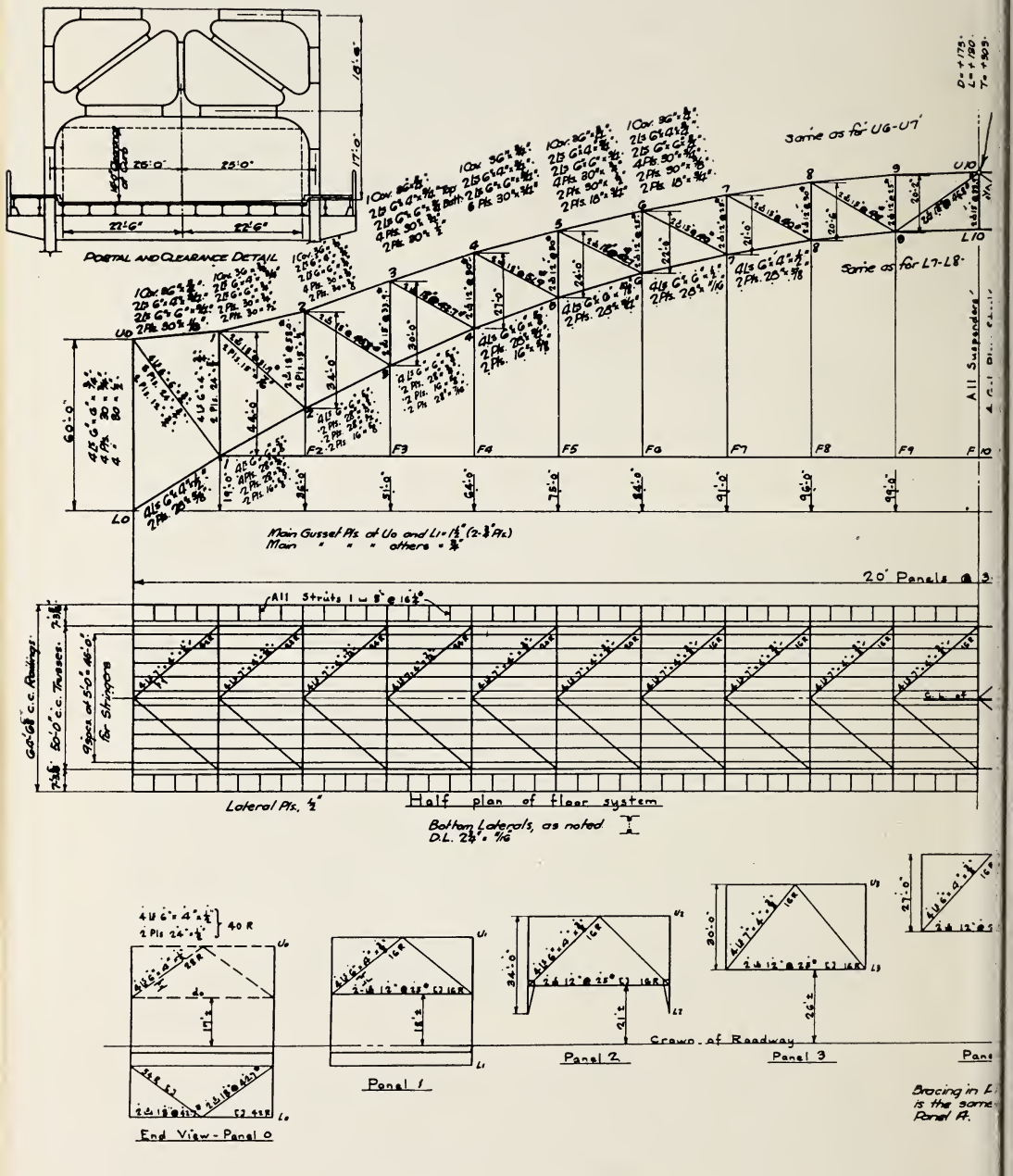
1. The cost of concrete is the same for all three solutions. With regard to Exhibits A and B, the cost of additional quantity of 200 cu. yd. of concrete in the counterweight is more than offset by savings effected through the use of lower and narrower piers.

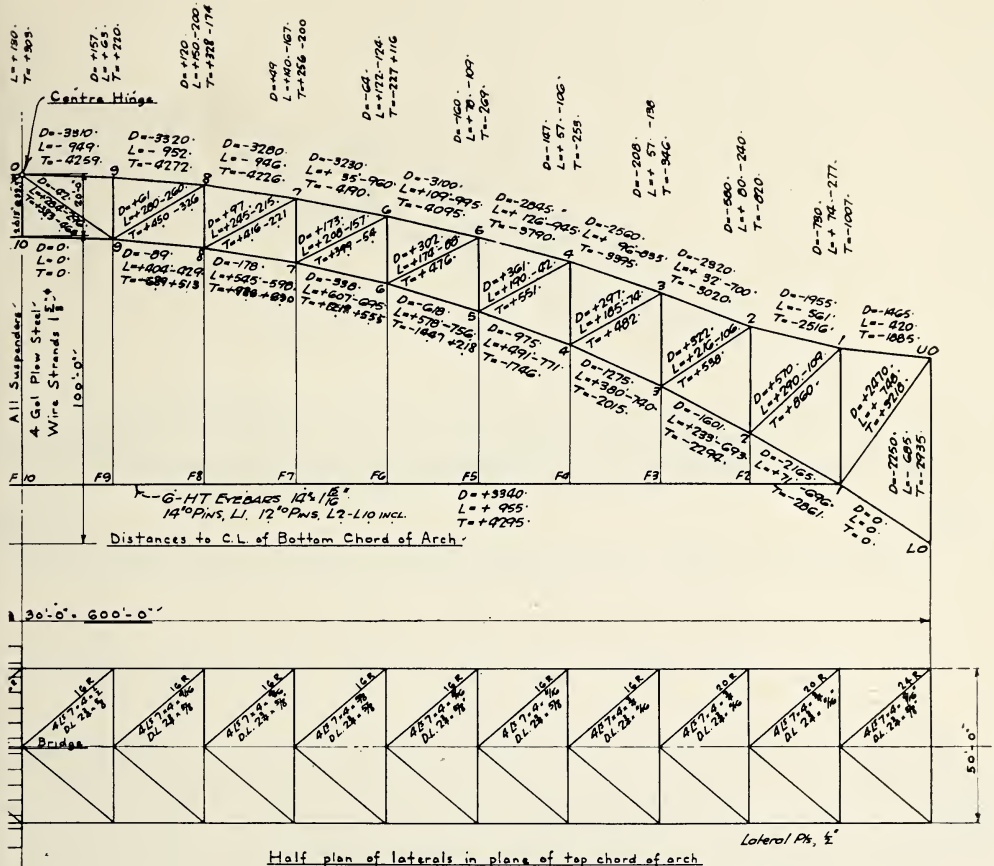
2. The cost of the steel reinforcement of the roadway is the same for all three solutions. In Exhibits A and B savings are effected on formwork and some steel; the conventional bridge, on the other hand, has no welding cost.

3. For erection the conventional trussed deck bridge is assumed to be 0.5 cents per pound cheaper than Exhibit A and 1.5 cents per pound cheaper than Exhibit B. The assumed erection methods are shown on Figure 63.

4. Bearings, expansion joints, railings, inspection walks, drainage, and other secondary items are omitted, being assumed equal for all three solutions.

"The estimates of costs have been established on the basis of


Portal and Clearance Detail
Same as for U6-U7
Same as for L7-L8
All Suspenders
Main Gusset Pls. at Uo and L11/2" (2-3/4" Pls.)
Main " " " others = 3/4"
20' Panels @ 3
All Struts 1 L 8" @ 16.8#
Lateral Pls. 1/2"
Half plan of floor system
Bottom Laterals, as noted
D.L. 2 3/4 = 5/16
End View - Panel 0
Panel 1
Panel 2
Panel 3
Crown of Roadway
Bracing in L is the same Panel A.



Half plan of laterals in plane of top chord of arch

Top Laterals, as noted
Top Struts, U1 to U9 incl, 4 L 6" x 4" x 5/8"
Top Strut of U10, 2 Struts of 4 L 6" x 4" x 5/8"

Lacing { Vert. DL 2 3/4" x 5/16", Horiz. SL 2 3/4" x 5/16", except at U1 DL 2 3/4" x 1/2"

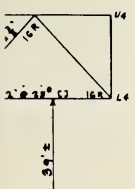
Rg. Stringers 21" WF Beam @ 73#

Int. Floorbeams Web Pl 54" x 3/8"
Each 2 L's 6" x 6" x 5/8"
Flange 1 Cov 16" x 3/4" 37'lg.

S.W. Stringers 21" WF Beam @ 73#

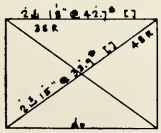
End Floorbeams at Piers 9 & 11
Web Pl 54" x 3/8"
Each 2 L's 6" x 6" x 5/8"
Flange 1 Cov 16" x 1/2" 39'lg.

End Floorbeams at Pier 10
Web Pl 54" x 3/8"
Each 2 L's 6" x 6" x 5/8"
Flange 1 Cov 16" x 3/4" 37'lg.



Panel 4

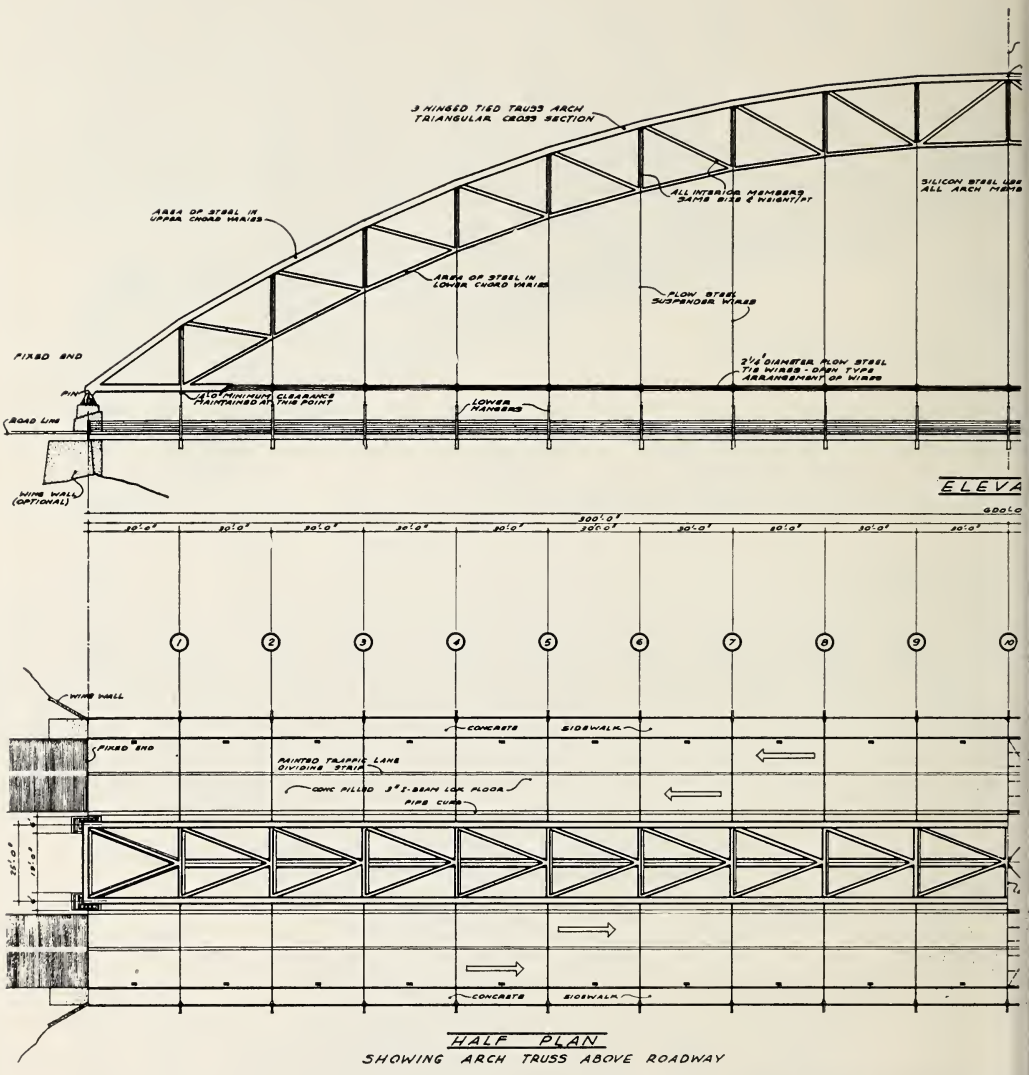
in Panels 5 to 10 same as shown for 4.



Bracing in End Panel in plane of Bottom Chord of Arch.

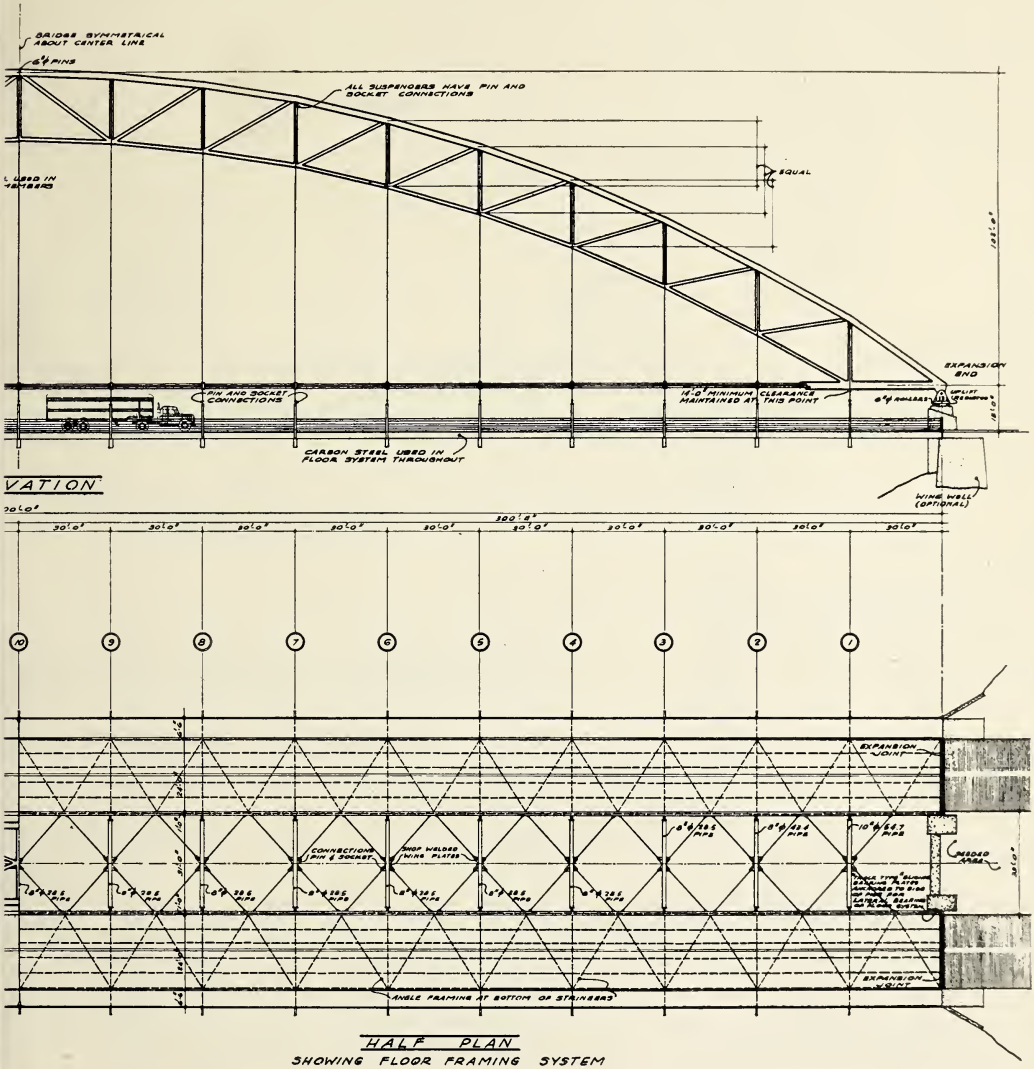
Main Material of arch members together with their gussets and splice material, Silicon Steel. Floor steel, bracing, detail material of arch members, Carbon Steel.
Live Load - M20 plus impact.
Unit Stresses - See Sh. 1 and Special Provisions
All stresses are in Kips.
Live Load Stresses shown hereon include Unif L.L., Concentrated L.L., Impact and Sidewalk stresses
Arches are symmetrical about center lines except end panels to keep end posts vertical.

Figure 65



HALF PLAN
SHOWING ARCH TRUSS ABOVE ROADWAY

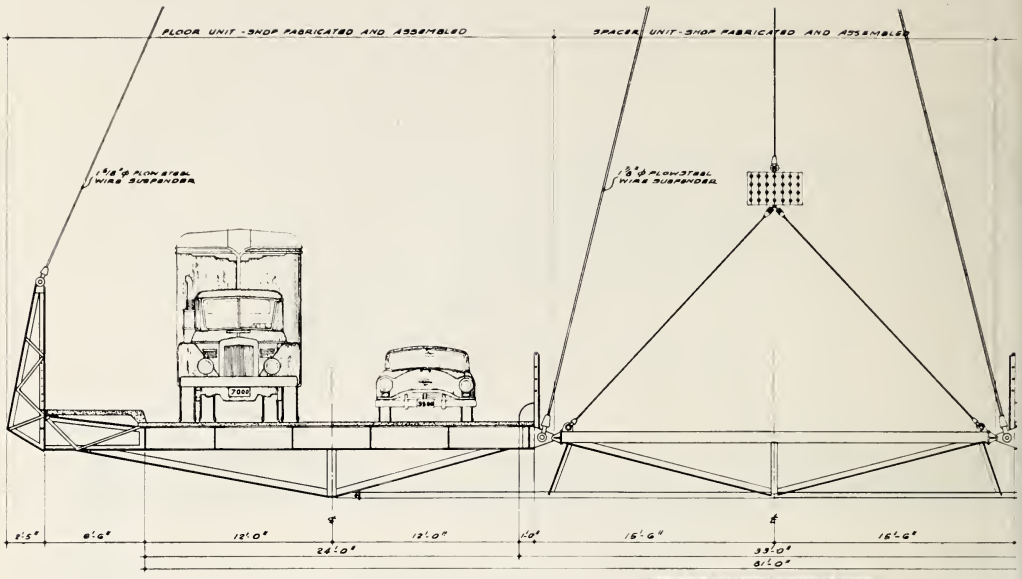
THREE HINGED TIED ARCH
CENTER SUPPORT TYPE • 60C



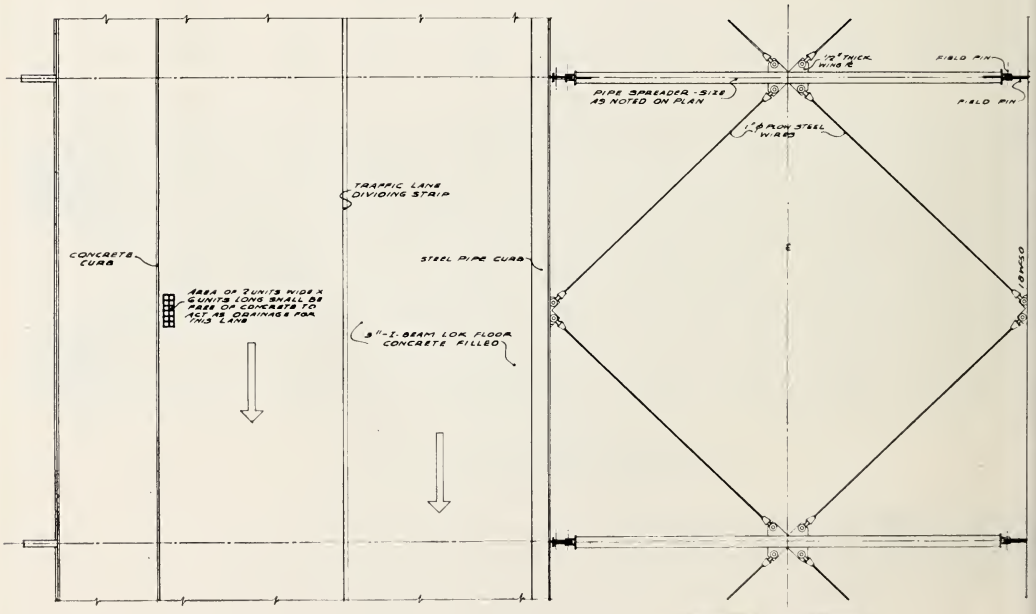
HALF PLAN
SHOWING FLOOR FRAMING SYSTEM

CH BRIDGE
500' SPAN

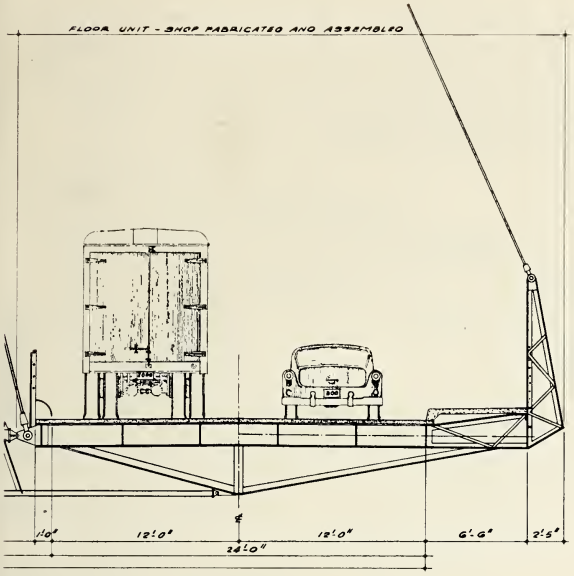
Figure 66



TYPICAL SECTION


TRAFFIC LANE DIVIDING STRIP
CONCRETE CURB
STEEL PIPE CURB
AREA OF 2 UNITS WIDE X 6 UNITS LONG SHALL BE FREE OF CONCRETE TO ACT AS DRAINAGE FOR THIS LANE
8"-I-BEAM LOK FLOOR CONCRETE FILLED
1/2" THICK HINGE
FIELD PIN
PIPE SPREADER - SIZE AS NOTED ON PLAN
FIELD PIN
1" ϕ PLOW STEEL WIRE
10'-0"

FLOOR PLAN



GENERAL PROVISIONS

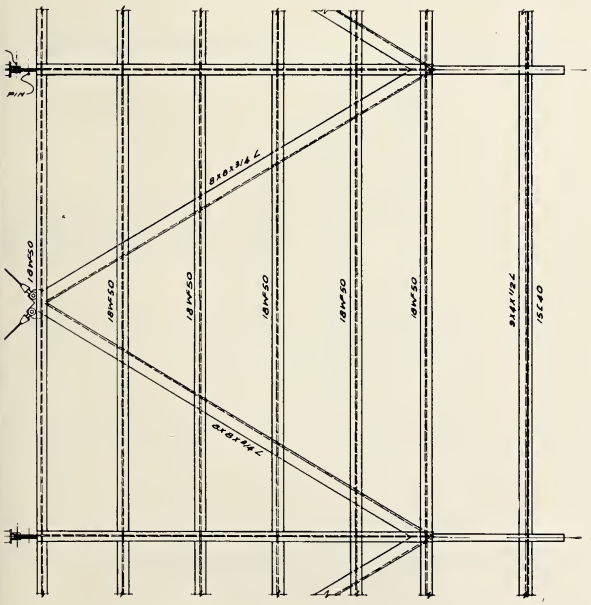
LIVE LOAD
640 LB. PER LINEAR FOOT PER 10 FOOT TRAFFIC LANE AND A SINGLE CONCENTRATED LOAD PER TRAFFIC LANE OF 24,000 LB FOR SHEAR. 18,000 LB. FOR MOMENT

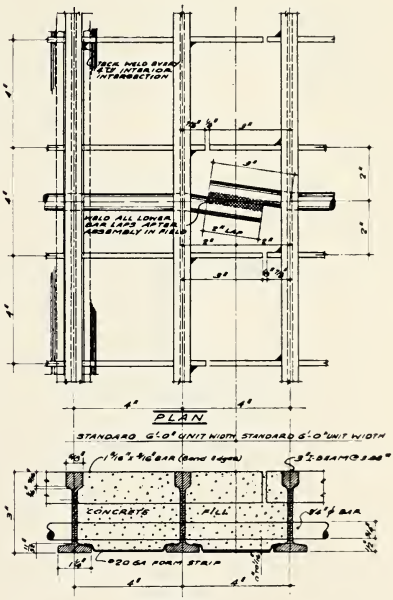
IMPACT = $\frac{50}{L+125}$

FLOOR BEAMS AND STRINGERS STANDARD H-20 LOADING IMPACT = 30 PERCENT

ALLOWABLE UNIT STRESSES POUNDS/SQ. IN.

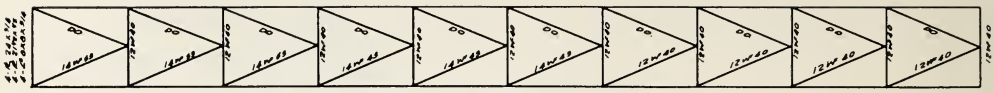
STEEL	CARBON	SILICON
TENSION	18,000	24,000
COMPRESSION	18,000 - 75 $\frac{L}{r}$	24,000 - 100 $\frac{L}{r}$
COMPRESSION - FLANGES OF BEAMS & GIRDERS	13,000 - 300 $\frac{L}{b}$	24,000 - 260 $\frac{L}{b}$
BEARING	27,000	36,000
SHEAR - WEB	11,250	16,000
SHEAR - PINS	15,500	
BENDING IN PINS	23,000	
BEARING ON MASONRY	600	



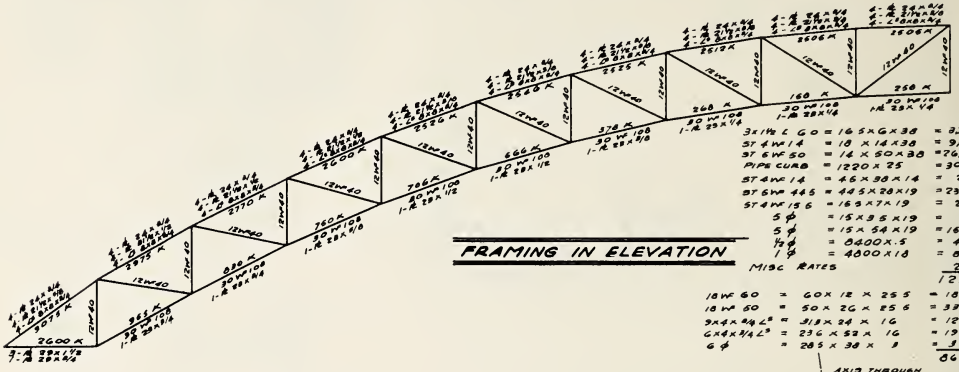


3" I-BEAM LOK FLOORING

Figure 67



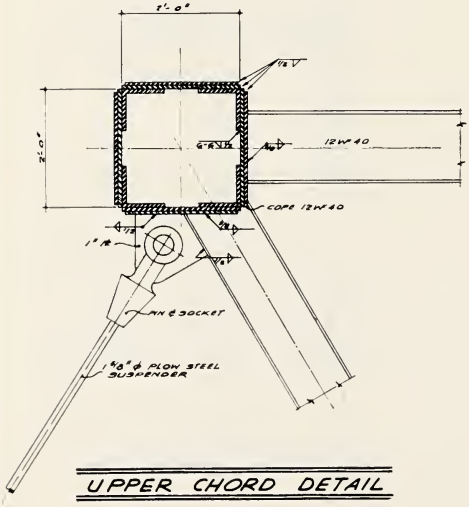
FRAMING IN PLANE OF UPPER CHORD



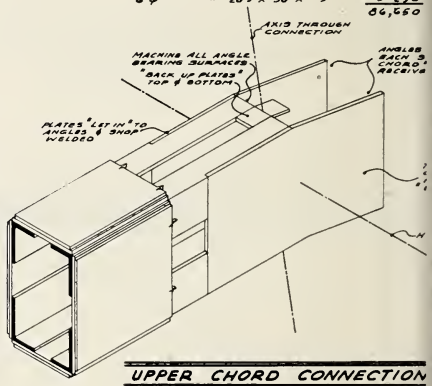
3x1½ L GO	= 16.5 x G x 38	= 3760
ST 4 WF 14	= 18 x 14 x 38	= 9570
ST 6 WF 50	= 14 x 50 x 38	= 26,500
PIPE CURB	= 1220 x 25	= 30,000
ST 4 WF 14	= 46 x 38 x 14	= 2390
ST 6 WF 44.5	= 44.5 x 28 x 19	= 23700
ST 4 WF 13.6	= 18.5 x 7 x 19	= 7060
5 ℄	= 15 x 35 x 19	= 960
5 ℄	= 15 x 54 x 19	= 16400
½ ℄	= 8400 x .5	= 4200
1 ℄	= 4800 x 18	= 8640
Misc Rates		2000
		129,160

18 WF 60	= 60 x 12 x 25.5	= 18 700
18 WF 60	= 50 x 26 x 25.6	= 33 100
9x4x 9/16 L°	= 31.3 x 24 x 16	= 12 000
6x4x 5/16 L°	= 23.6 x 52 x 16	= 19 600
6 ℄	= 283 x 38 x 3	= 3 260
		84,660

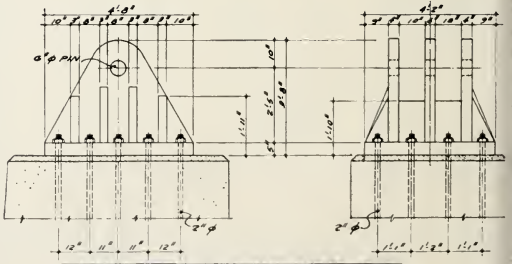
FRAMING IN ELEVATION



UPPER CHORD DETAIL



UPPER CHORD CONNECTION



TYPICAL FIXED END BEARING

24 x ¾ x 87 = 6/12 x 4 x 4 x 87 = 37,000
2 1/16 x 5/8 x 37 = 45.7 x 4 x 4 x 37 = 27,000
8 x 8 x 8 x 3/4 x 37 = 36.9 x 4 x 4 x 37 = 25,000
89,000

24 x ¾ x 36 = 6/12 x 4 x 4 x 36 = 34,600
2 1/16 x 1/2 x 36 = 36.6 x 4 x 4 x 36 = 20,600
8 x 8 x ¾ x 36 = 36.9 x 4 x 4 x 36 = 21,400
76,600

24 x ¾ x 34 = 6/12 x 4 x 4 x 34 = 33,100
2 1/16 x 1/2 x 34 = 36.6 x 4 x 4 x 34 = 13,900
8 x 8 x ¾ x 34 = 36.9 x 4 x 4 x 34 = 21,200
74,600

24 x 3/4 x 33 = 6/12 x 4 x 4 x 33 = 32,800
2 1/16 x ¾ x 33 = 27.4 x 4 x 4 x 33 = 14,400
8 x 8 x ¾ x 33 = 36.9 x 4 x 4 x 33 = 20,500
67,200

24 x ¾ x 32 = 6/12 x 4 x 4 x 32 = 32,800
2 1/16 x ¾ x 32 = 27.4 x 4 x 4 x 32 = 14,400
8 x 8 x ¾ x 32 = 36.9 x 4 x 4 x 32 = 20,500
67,200

24 x ¾ x 32 = 6/12 x 4 x 4 x 32 = 31,400
2 1/16 x ¾ x 32 = 27.4 x 4 x 4 x 32 = 14,000
8 x 8 x ¾ x 32 = 36.9 x 4 x 4 x 32 = 19,800
65,200

24 x ¾ x 31 = 6/12 x 4 x 4 x 31 = 30,400
2 1/12 x ¾ x 31 = 27.4 x 4 x 4 x 31 = 13,600
8 x 8 x ¾ x 31 = 36.9 x 4 x 4 x 31 = 19,300
63,300

18 WF 50 = 60 x 12 x 600 = 360,000
15 C 40 = 40 x 2 x 600 = 48,000
9 x 4 x 7/16 = 21.3 x 2 x 600 = 25,600
8 x 8 x 1/2 = 36.9 x 4 x 20 x 13 = 96,000
$529,600

10 ϕ - 54.7 x 28 x 2 = 3060
8 ϕ - 43.4 x 28 x 2 = 2430
8 ϕ - 28.5 x 28 x 15 = 11950
17,440

9 x 4 x 5/16 L² = 21.3 x 24 x 12 = 9000
6 x 4 x 5/16 L² = 23.6 x 52 x 12 = 14700
23,700

24 x ¾ x 30 = 6/12 x 4 x 4 x 30 = 29400
2 1/12 x ¾ x 30 = 27.4 x 4 x 4 x 30 = 19 300
8 x 8 x ¾ 4 L x 30 = 36.9 x 4 x 4 x 30 = 16 600
61 300

24 x ¾ x 30 = 6/12 x 4 x 4 x 30 = 29400
2 1/16 x ¾ x 30 = 27.4 x 4 x 4 x 30 = 19 900
8 x 8 x ¾ 4 L x 30 = 36.9 x 4 x 4 x 30 = 16 600
61 900

12 WF 40 = 38 x 40 x 29 = 35 000
12 WF 40 = 4 x 40 x 300 = 48 000
83 000

24 x 5/8 x 28 = 6/12 x 4 x 2 x 28 = 11 760
2 1/12 x ⅝ x 28 = 27.4 x 4 x 2 x 28 = 6 040
8 x 8 x ¾ 4 L x 29 = 36.9 x 4 x 2 x 29 = 7 160
23 450

7 - 1 1/2 x 29 x 46 = 2 x 7 x 46 x 146 = 99 500
12 WF 40 = 19 x 40 x 23 = 17 500
12 WF 40 = 16 x 40 x 32 = 20 800
14 WF 43 = 12 x 43 x 33 = 17 100
14 WF 63 = 12 x 53 x 33 = 21 000
75 900

SOCKETS
2 1/4 ϕ - 56 x 166 = 9260
1 7/8 ϕ - 38 x 85 x 2 = 6460
1 4/8 ϕ - 38 x 55 x 2 = 4180
1 ϕ - 52 x 16.5 = 806
⅛ ϕ - 102 x 225 = 230
20 926

6 WF 16.5 = 12 x 12 x 15.5 = 3160
6 C 13 = 4 x 12 x 13 = 624
1 ¼ ϕ x 3' = 7 x 3.6 x 17 = 398
4442

30 WF 108 x 83 = 108 x 83 x 2 = 7140
25 x ¾ x 85 = 587 x 85 x 2 = 3870
11 010

30 WF 108 x 84 = 108 x 84 x 2 = 7340
29 x ¾ x 34 = 887 x 34 x 2 = 3860
11 200

30 WF 108 x 98 = 108 x 98 x 2 = 7140
28 x ⅝ x 98 = 409 x 98 x 2 = 3220
10 360

30 WF 108 x 92 = 108 x 92 x 2 = 6920
29 x ½ x 32 = 391 x 32 x 2 = 2500
9 420

30 WF 108 x 92 = 108 x 92 x 2 = 6920
28 x ½ x 92 = 391 x 32 x 2 = 2600
9 520

30 WF 108 x 91 = 108 x 91 x 2 = 6700
29 x ⅝ x 91 = 293 x 31 x 2 = 1820
8 520

30 WF 108 x 90 = 108 x 30 x 2 = 6490
29 x ¼ x 90 = 196 x 30 x 2 = 1180
7670

30 WF 108 x 90 = 108 x 30 x 2 = 6490
29 x ½ x 90 = 196 x 30 x 2 = 1180
7670

2 ¼ ϕ - 510 x 28 x 10 = 142800
1 ⅞ ϕ - 600 x 4 x 6.3 = 15100
1 4/8 ϕ - 790 x 4 x 4.76 = 15000
1 ϕ - 76 x 17 x 1.8 = 2820
⅛ ϕ - 1520 x .5 = 760
I-BEAM LOK FLOOR 175,380
25.5 x 1200 x 16.5 = 474,000

60 70 100 000 990 100 160 300 100 100 140 100 180 100 100 100 200 600 160 560

SIZES ON LOWER END OF CH. SUCCESSIVE UPPER CHORDS CUT TO GIVE PROJECTING PLATES

THESE PLATES FACILITATE GUIDANCE OF THE UPPER CHORDS INTO PLACE AS WELL AS PROVIDING BACKING UP FOR THE SUBSEQUENT BUTT FIELD WELDING

SUMMARY

EXHIBIT "A"		EXHIBIT "B"	
I-BEAM LOK FLOOR	465,000 Lb.	I-BEAM LOK FLOOR	474,000 Lb.
STRUCTURAL (SILICON STEEL)	1,525,000 Lb.	STRUCTURAL (SILICON STEEL)	1,046,740 Lb.
STRUCTURAL (CARBON STEEL)	2,250,000 Lb.	STRUCTURAL (CARBON STEEL)	315,157 Lb.
STEEL CASTINGS	172,400 Lb.	SUSPENDERS & TIE WIRES	175,980 Lb.
SUSPENDER STRANDS	31,500 Lb.	SOCKETS (ALL SIZES)	20,926 Lb.
BRIDGE RAILING	15,000 Lb.		
EYE BARS & PINS	720,000 Lb.		
	5,189,900 Lb.		2,632,803 Lb.

NOTE: THESE QUANTITIES OBTAINED FROM SHEET NO 1 (TITLE SHEET) OF EXHIBIT "A".

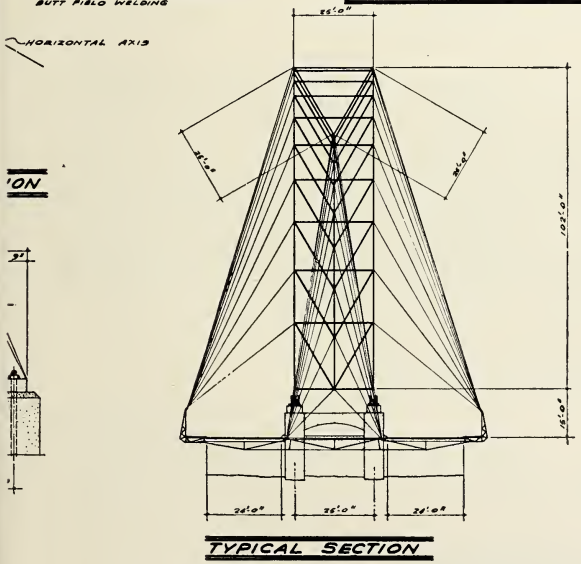
5,189,900 Lb. = 2594.9 TONS

NOTE: THESE QUANTITIES ARE A SUMMATION OF THE BREAKDOWN LISTED ABOVE.

2,632,803 Lb. = 1316.4 TONS

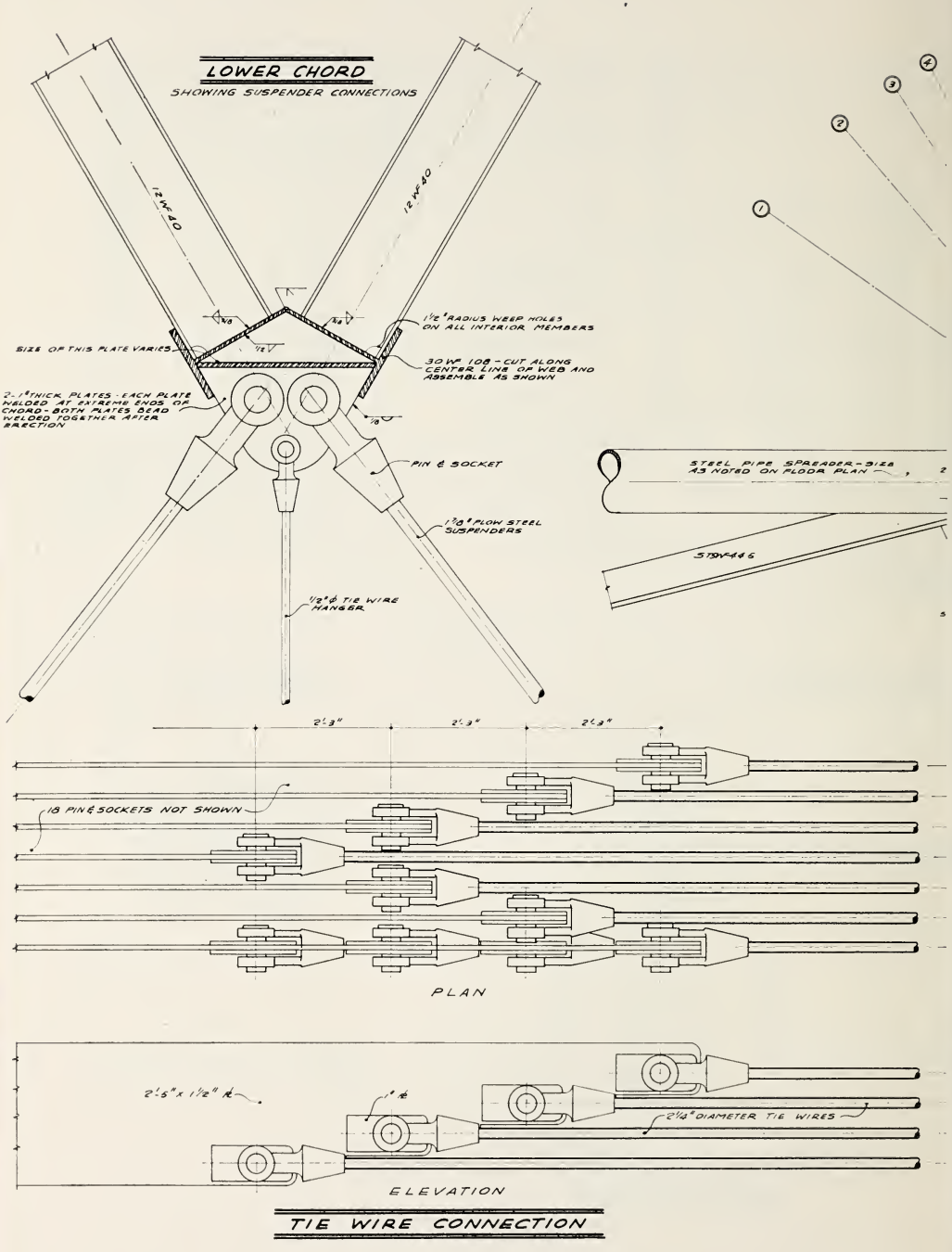
PERCENTAGE OF STEEL SAVING 49.2 %

WELDS			
SHOP		FIELD	
1/4	270	1/4	964
5/16	260	5/16	2071
3/8	18,343	1/2	1380
TOTAL 18,873 Lb.		TOTAL 4264 Lb.	
WEIGHT OF WELDS PER TON OF STRUCTURAL STEEL = 17.64 Lb.			



TYPICAL SECTION

Figure 68


LOWER CHORD
SHOWING SUSPENDER CONNECTIONS
12 WF 40
12 WF 40
1½" RADIUS WEEP HOLES ON ALL INTERIOR MEMBERS
SIZE OF THIS PLATE VARIES
30 WF 108 - CUT ALONG CENTER LINE OF WEB AND ASSEMBLE AS SHOWN
2 - 1" THICK PLATES - EACH PLATE WELDED AT EXTREME ENDS OF CHORD - BOTH PLATES DEAD WELDED TOGETHER AFTER ERECTION
PIN & SOCKET
1⅞" FLOW STEEL SUSPENDERS
½" TIE WIRE HANGER
STEEL PIPE SPREADER - SIZE AS NOTED ON FLOOR PLAN
STRUCTURAL
2'-3"
2'-3"
2'-3"
18 PIN & SOCKETS NOT SHOWN
PLAN
2'-5" x 1½"
1"
2½" DIAMETER TIE WIRES
ELEVATION

TIE WIRE CONNECTION

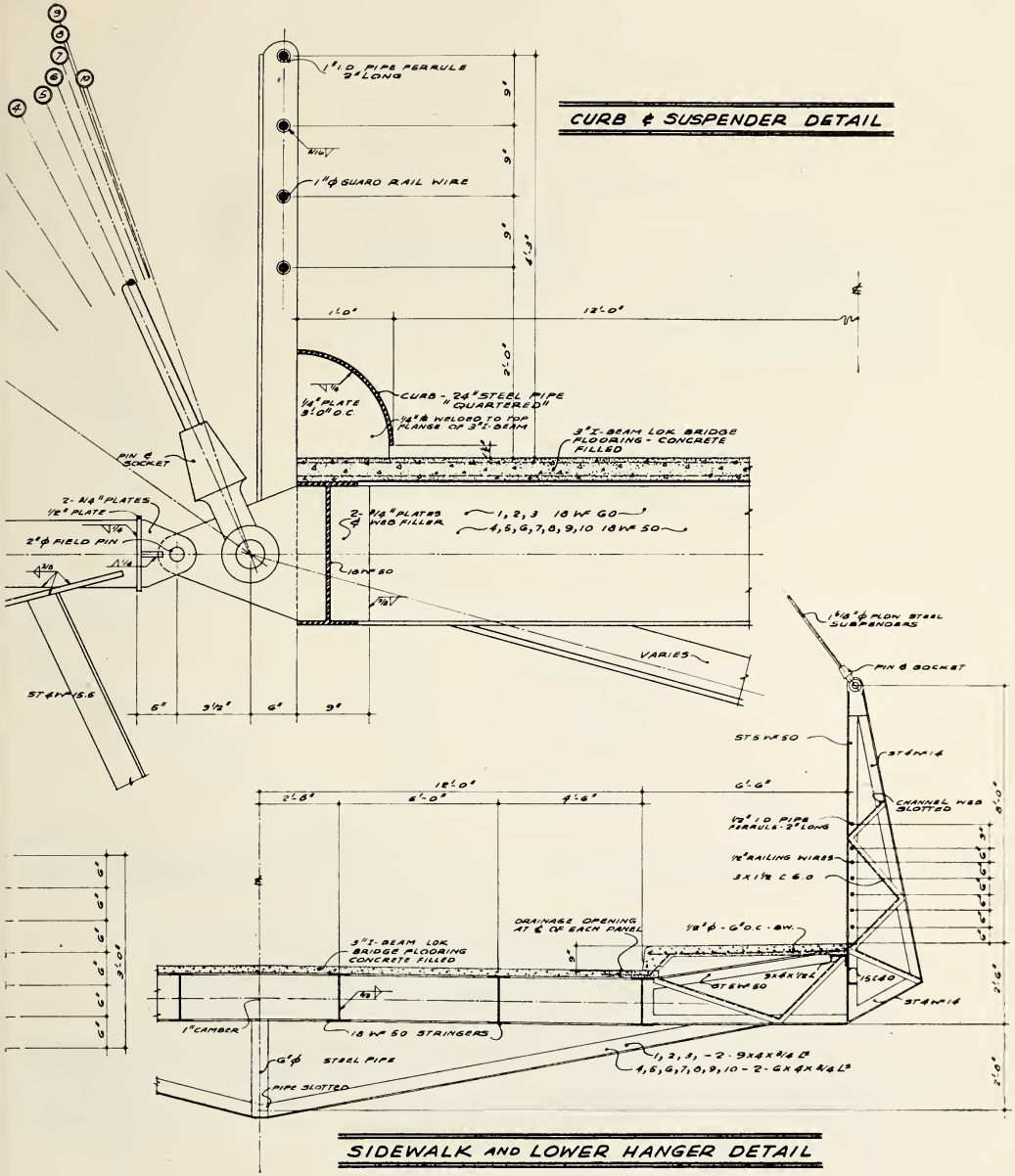
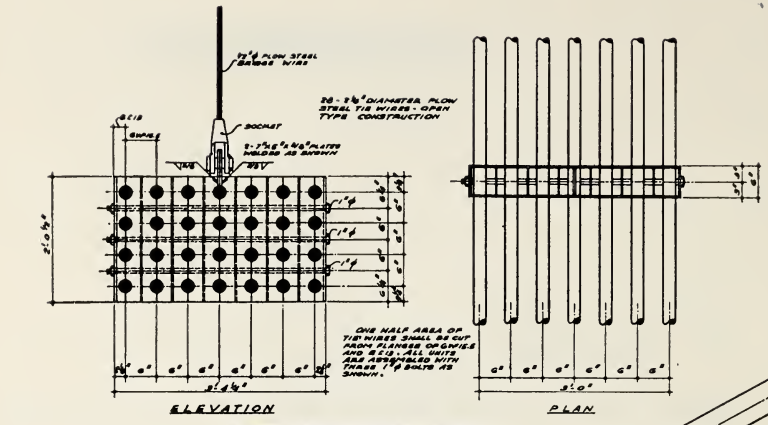
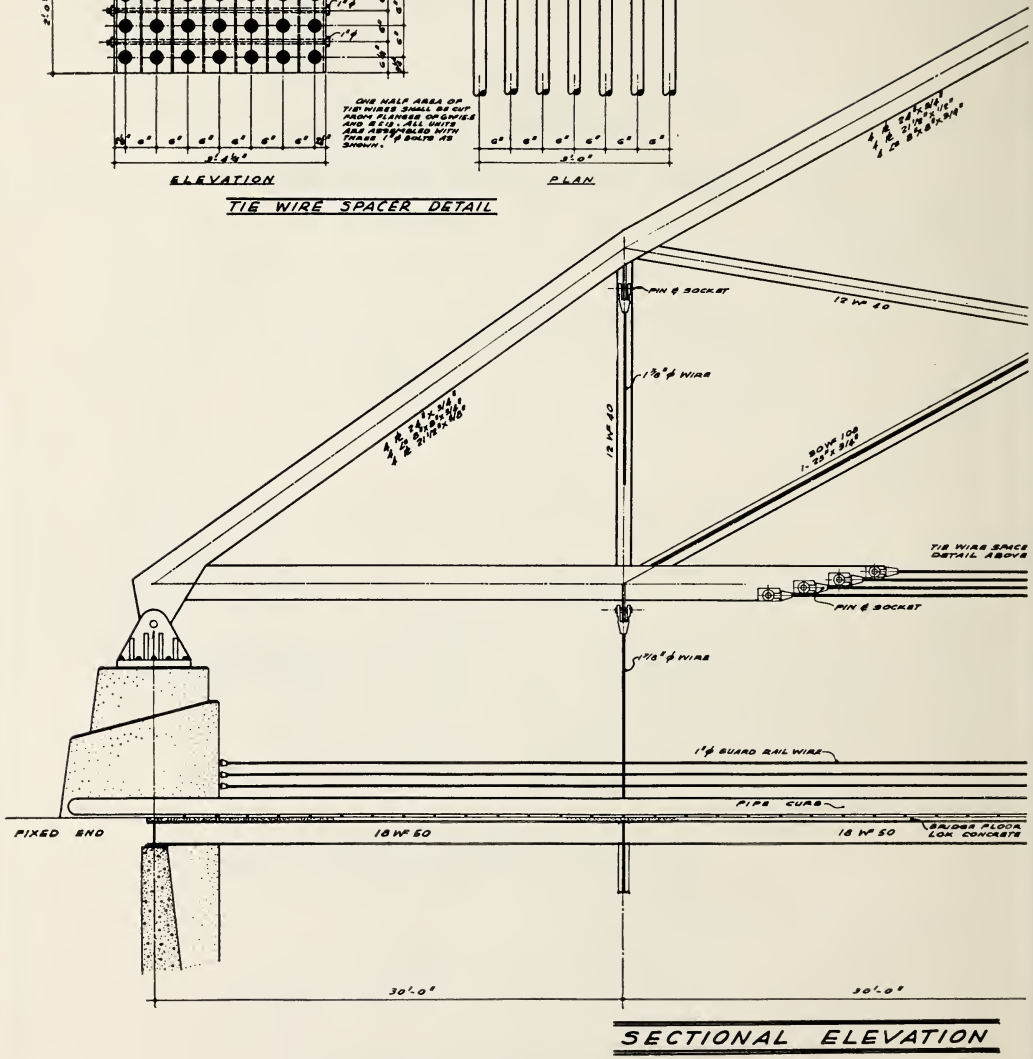

CURB & SUSPENDER DETAIL
1" I.D. PIPE FERRULE 2" LONG
1" ϕ GUARD RAIL WIRE
CURB - 24" STEEL PIPE "QUARTERED"
1/4" PLATE 3'-0" O.C.
1/4" ϕ WELDED TO TOP FLANGE OF 3" I-BEAM
3" I-BEAM LOK BRIDGE FLOORING - CONCRETE FILLED
PIN & SOCKET
2-2 1/4" PLATES
1/2" PLATE
2" ϕ FIELD PIN
2-1/2" PLATES AND FILLER
1, 2, 3 18 WF 60
4, 5, 6, 7, 8, 9, 10 18 WF 50
18 WF 50
VARIES
1 1/8" ϕ PLOW STEEL SUSPENDERS
PIN & SOCKET
ST 5 WF 50
ST 4 WF 14
CHANNEL WEB SLOTTED
1/2" I.D. PIPE FERRULE - 2" LONG
1/2" RAILING WIRES
3 X 1 1/2 C 6.0
DRAINAGE OPENING AT ℄ OF EACH PANEL
7/8" ϕ - 6'-0" O.C. - STAG.
3" I-BEAM LOK BRIDGE FLOORING CONCRETE FILLED
ST 5 WF 50
15 LB 40
1" CAMBER
18 WF 50 STRINGERS
6" ϕ STEEL PIPE
PIPE SLOTTED
1, 2, 3, - 2 - 9 X 4 X 7/16 L
4, 5, 6, 7, 8, 9, 10 - 2 - 6 X 4 X 7/16 L
SIDEWALK AND LOWER HANGER DETAIL

Figure 69



TIE WIRE SPACER DETAIL



SECTIONAL ELEVATION

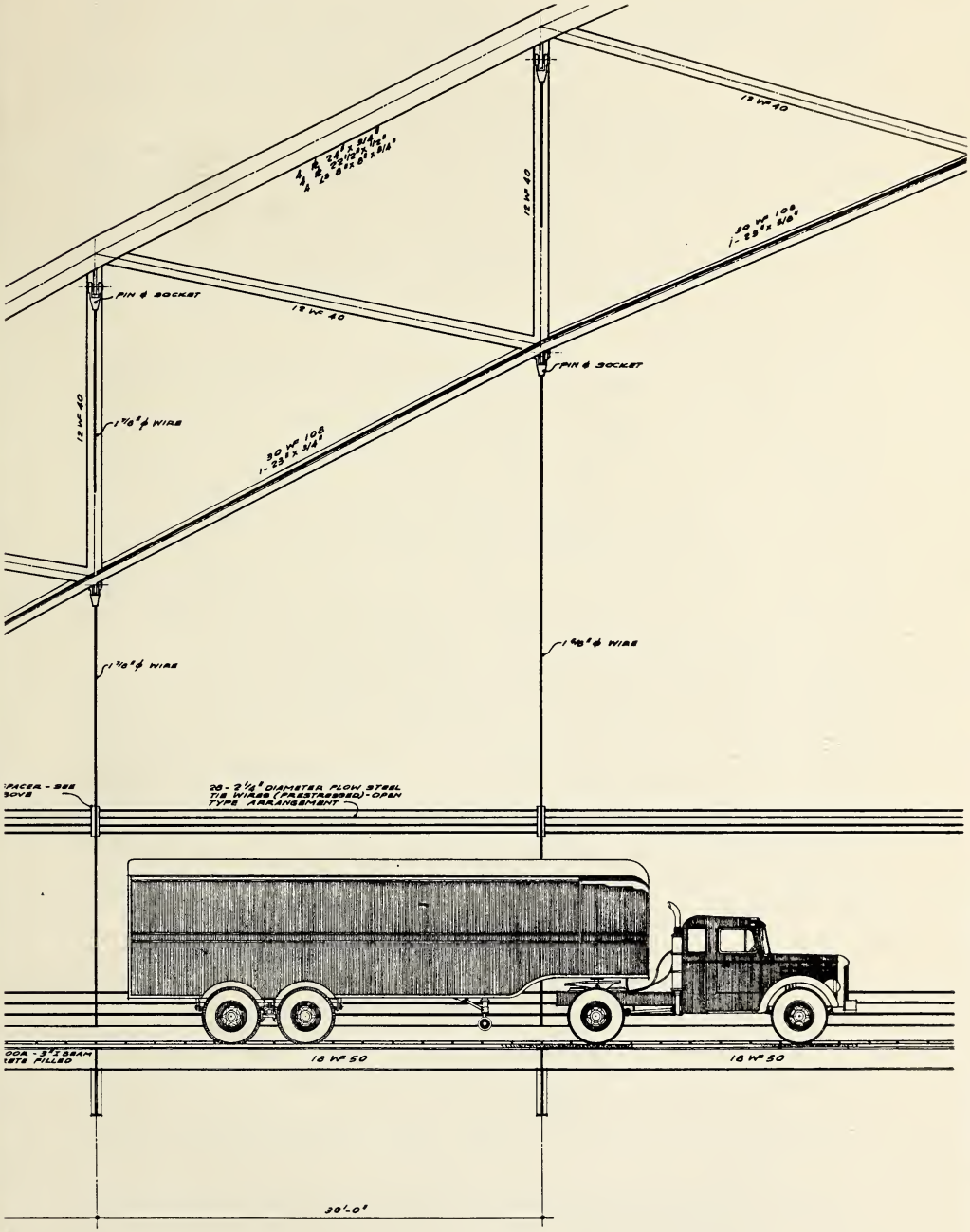

18 WF 70
12 WF 40
PIN & SOCKET
18 WF 40
PIN & SOCKET
12 WF 40
1 7/8" ϕ WIRE
30 WF 108
1 - 23" x 3/4"
1 7/8" ϕ WIRE
1 7/8" ϕ WIRE
SPACER - SEE ABOVE
28 - 2 1/4" DIAMETER FLOW STEEL TIE WIRES (PRESTRESSED) - OPEN TYPE ARRANGEMENT
18 WF 50
18 WF 50
30'-0"

Figure 70

the writer's own long experience in design and shopwork, as well as on a knowledge of the estimating of several contractors. The estimates of contractors vary considerably so that a breakdown of costs would be of restricted value and would furnish no essential contribution to the purpose of this entry."

K. W. Dobert, Cleveland, Ohio, used the riveted bridge shown in Figure 65 as his Exhibit A. This span plus another just like it are part of a total bridge crossing of 3420 ft. that has already been constructed. As the drawing indicates, these trussed arches are 50 ft. apart to provide 45 ft. of roadway. They have a span of 600 ft. and rise 100 ft. at mid-span. The 600 ft. of length is divided into twenty 30 ft. panels.

An elevation and plan of the welded arch are given in Figure 66 and a floor plan and typical section in Figure 67. For the welded bridge, there is a single triangular, trussed arch as indicated in the typical section of Figure 68. Each of the two top chords are built-up sections 2 ft. wide and 2 ft. deep as shown in the detail on Figure 68. The one bottom chord is basically triangular in shape (see Figure 69) that consists of the two tees, made by cutting a 30 WF 108, plus a plate 23 in. wide. The other members of this triangular, trussed arch are shown on the elevation of Figure 70. The triangle is equilateral—each side measuring 25 ft.

The welded arch has the same twenty panels of 30 ft. for a total length of 600 ft. It has a rise of 102 ft. The two 24 ft. roadways are separated by a distance of 33 ft. The trussed floorbeams at each panel point are shown in Figure 67. The twenty-eight tie wires (each 2¼ in. in diameter) for the tie member of the arch bridge are located along the centerline of the bridge.

A summary of the weights for both Exhibit A and Exhibit B are a part of Figure 68. Mr. Dobert gave the following advantages for Exhibit B:

(1) Compared with Exhibit A, Exhibit B saves 49.2 per cent of steel required for same function.
(2) Easy field erection.
(3) Wind bracing eliminated due to space frame arch.
(4) All arch members are generally the same size and relatively light in weight.
(5) Built-up members in the arch utilizing simple, linear welds.
(6) All of the arch members are confined. Lower chord forms a walkway where paint spray lines, ropes and welding cables are slung.

(7) The snow plow can "edge" the snow over to the center of the bridge where it will fall over the pipe curb. The blade can be raised to plow the sidewalk also.

(8) Divides the traffic lanes.

Walter Preimats, Asheboro, North Carolina, presented the design of a stiffened, tied arch. For the 200 ft. span (ten panels of 20 ft.), the rise is 35 ft. The arches are 30 ft. apart and carry a 26 ft. roadway as shown in Figure 71.

The arch ribs and the top chord bracing members are pipes. The ribs are 18 in. pipes and the bracing members are 8 in. pipes, arranged as K bracing.

The stiffening girder serves also as a tie. It consists of a 60 in. web with two flanges. Each flange has one 6 x 6 in. angle (rotated) and a 16 in. plate. Details of this girder are given in Figure 72.

The 5½ in. reinforced concrete slab is supported by 6 lines of stringers that are spaced at 5 ft.-3 in. centers. The interior stringers are 18 WF 50 and the curb stringers are 16 WF 36. The floorbeams are 33 WF 130. These members are shown on Figure 73.

Details of the arch pipes and hangers (1⅞ in. rounds) are shown in Figure 74.

In discussing his design, Mr. Preimats said:

"Advantages of such a girder section are as follows:

(1) There are separated welds closer to the edge of the flange plate instead of welds concentrated at the middle of the flange plate.

(2) It is possible to build-up such girder sections without specially rolled shapes.

(3) There is more inner connection between web and flange plate and a better distribution of stresses than in direct web-to-plate welded girders.

(4) Deformation of flange plate is prevented.

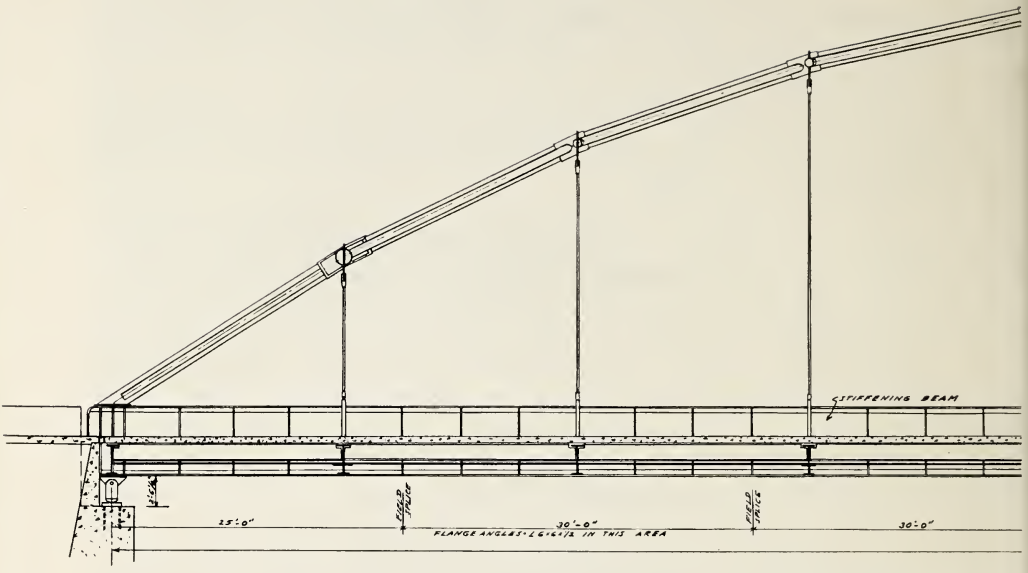
(5) Flange under compression is better stiffened in horizontal direction.

(6) Height of web plate is lower. It is possible to use a thinner web plate and under certain circumstances less stiffeners.

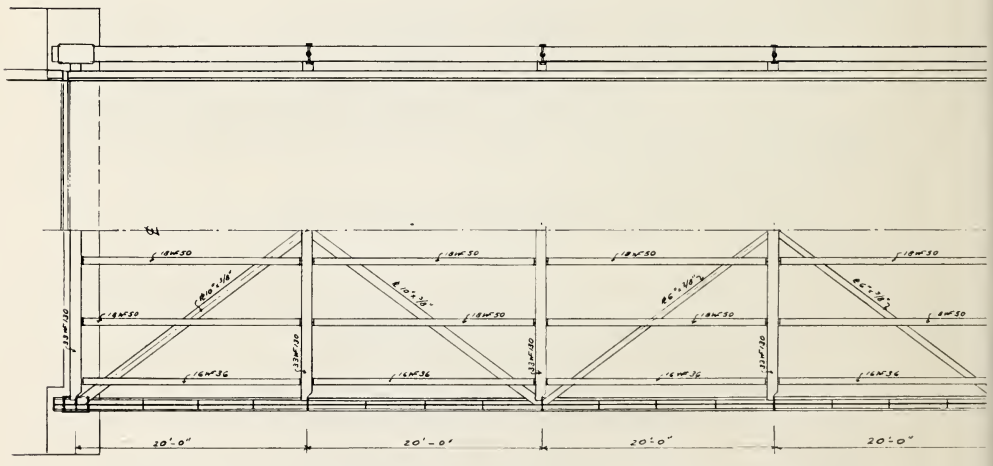
(7) Making-the-welds heat is not concentrated in one spot but more distributed over the flange area.

(8) It is believed that the appearance of this girder is better than that of a girder built-up in the usual way.

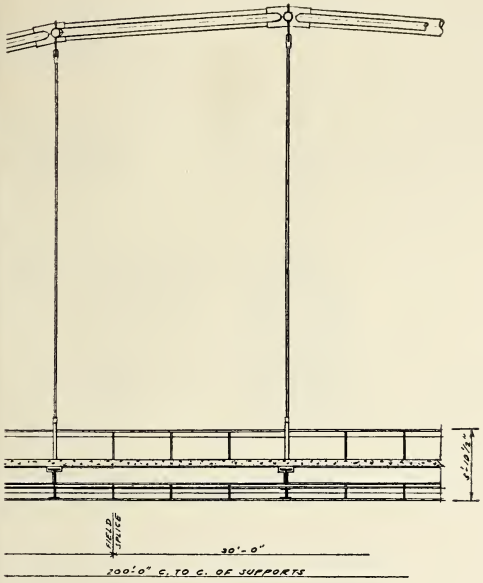
"There is no danger of rust if the inside faces of angles and plate are coated before assembling and the ends of angles are sealed because neither air nor moisture can enter this closed space.



- LONGITUDINAL SECTION



- PLAN - SECTION



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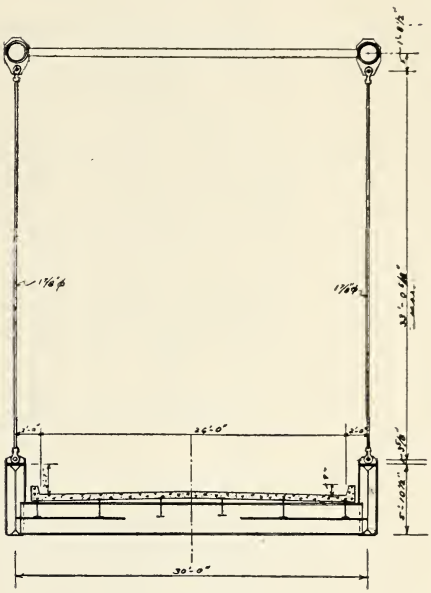

TRANSVERSE SECTION

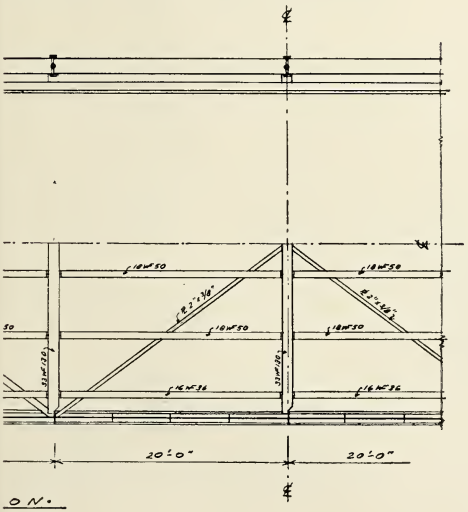

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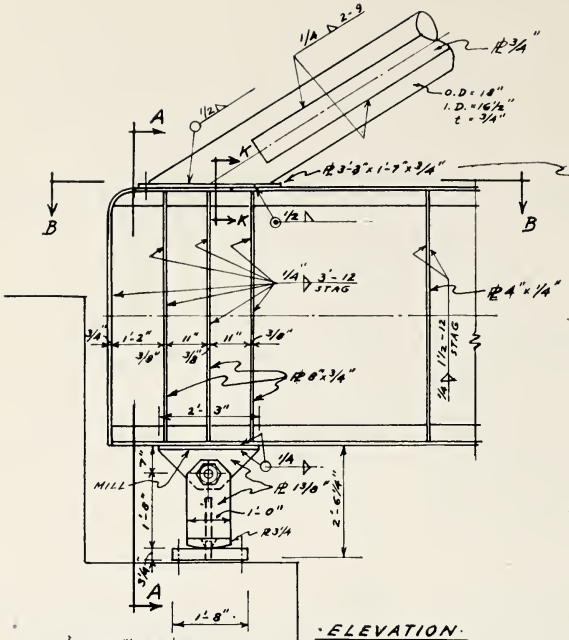
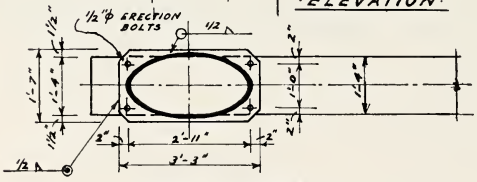
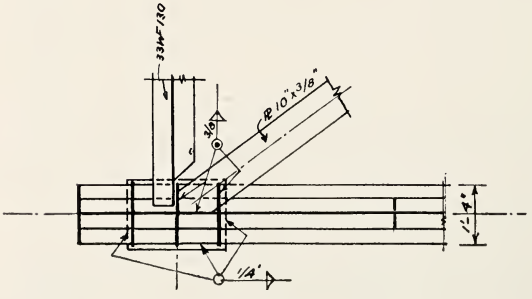
Figure 71



·ELEVATION·

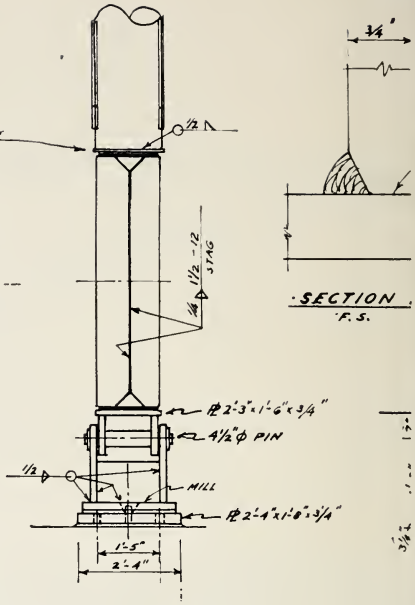


·SECTION B-B·



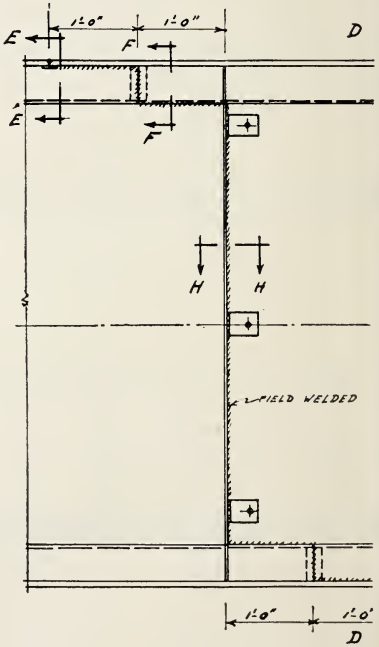
·SECTION C-C·

·DETAILS OF STIFFENING BEAM AND EXPANSION BEARING·

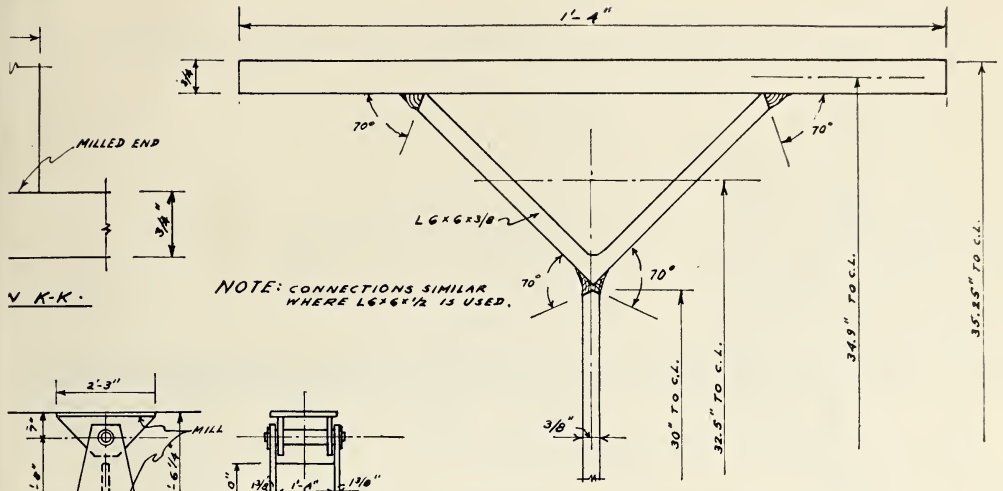


·SECTION A-A·

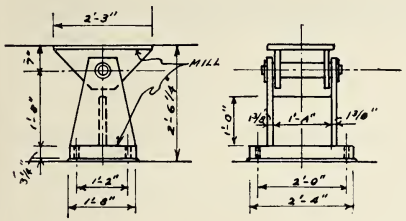
·SECTION F.S.



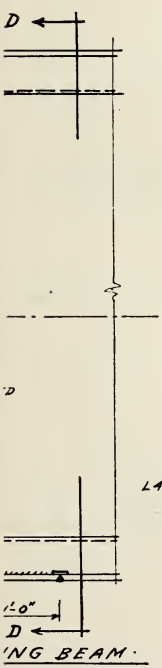
·FIELD SPLICE OF STIFFENIN



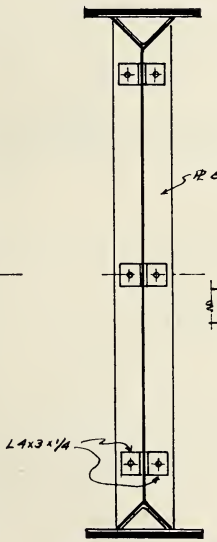
DETAIL OF WELDED CONNECTIONS OF STIFFENING BEAM.



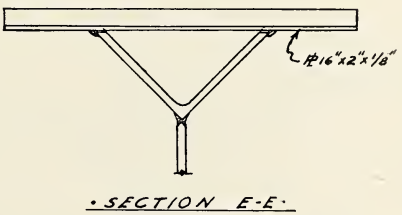
·FIXED BEARING·



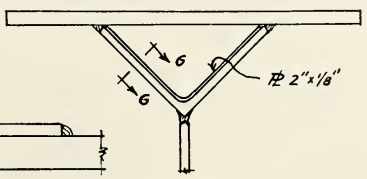
'NG BEAM·



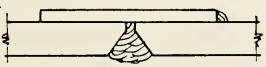
·SECTION D-D·



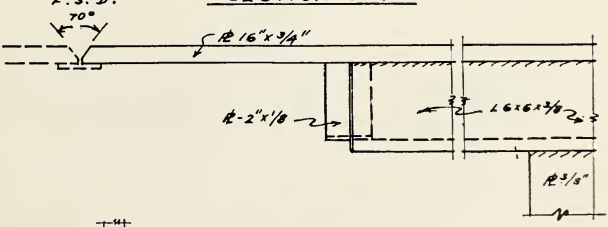
·SECTION E-E·



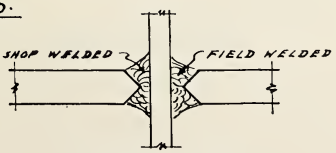
·SECTION F-F·



·G-G·
F.S.D.

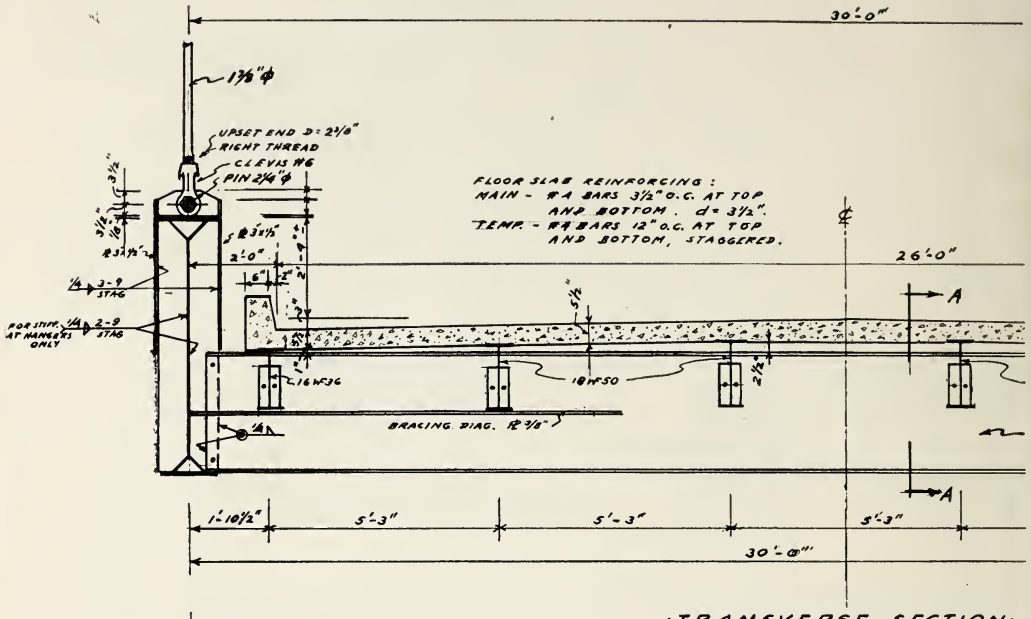


·DETAIL OF FIELD SPLICE·

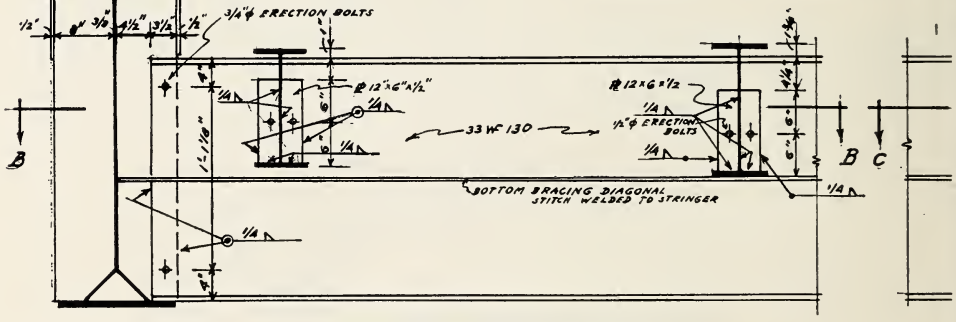


·SECTION H-H·
F.S.D.

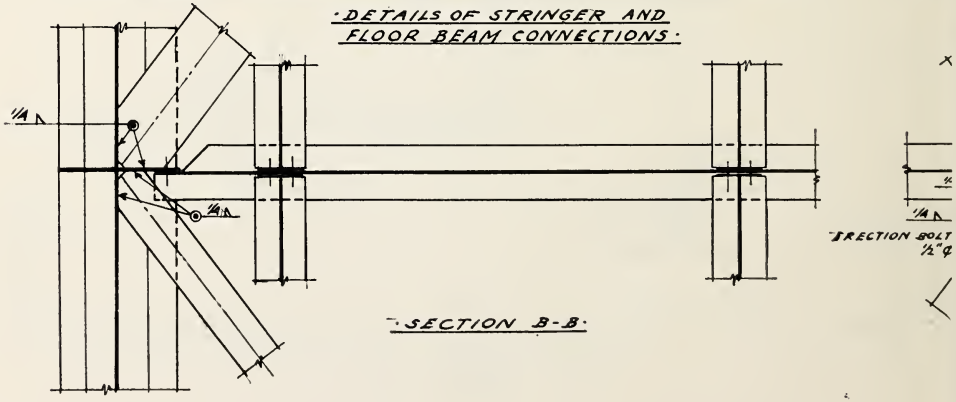
Figure 72



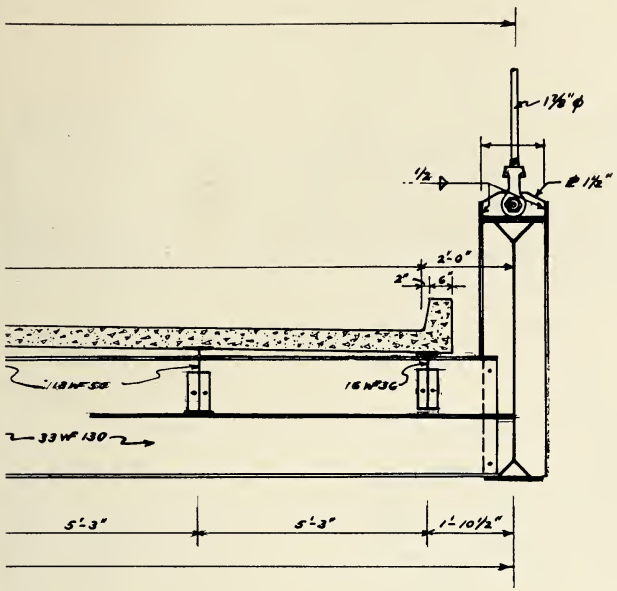
·TRANSVERSE SECTION·

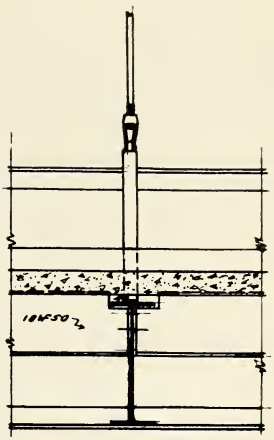


·DETAILS OF STRINGER AND FLOOR BEAM CONNECTIONS·



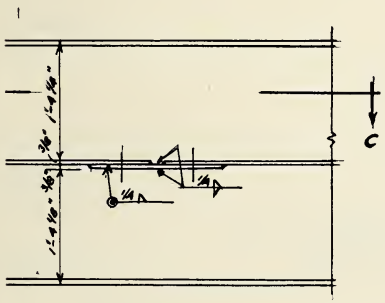
·SECTION B-B·

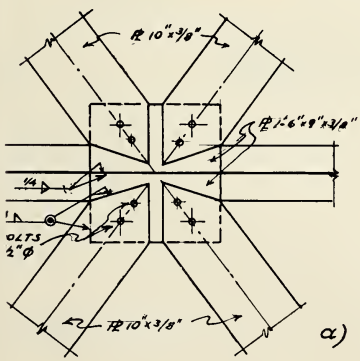


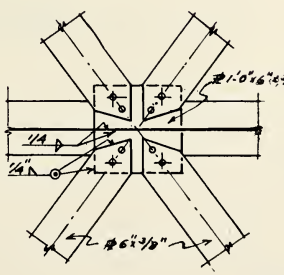


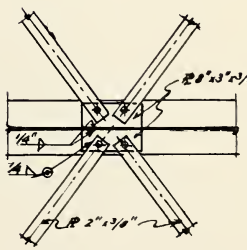
·SECTION A-A·

1·



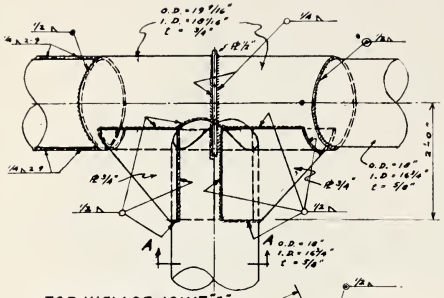




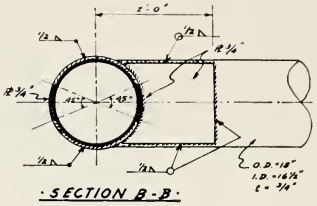


·SECTION C-C·

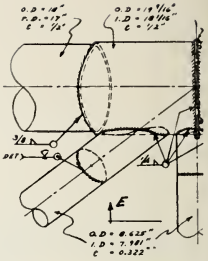
Figure 73



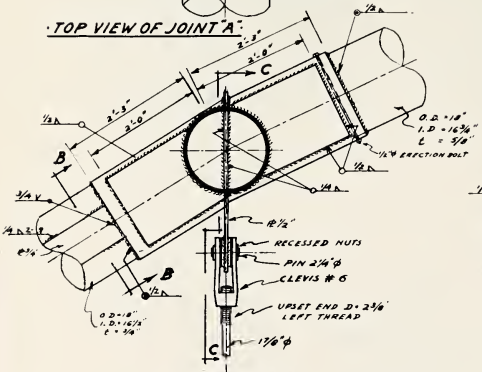
·TOP VIEW OF JOINT "A"·



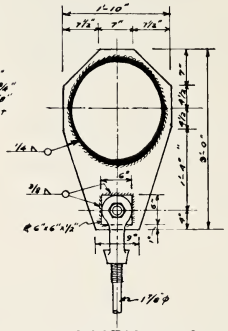
·SECTION B-B·



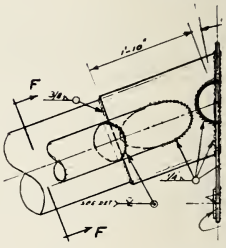
·TOP VIE



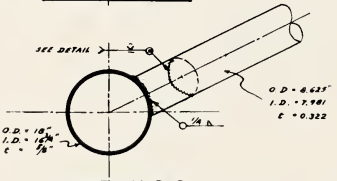
·SECTION A-A·



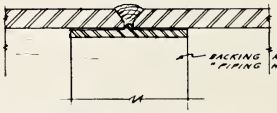
·SECTION C-C·



·SECTI

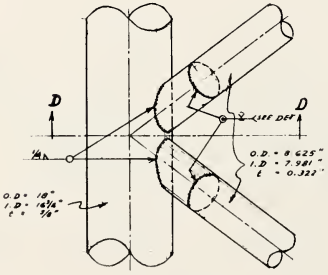


·SECTION D-D·

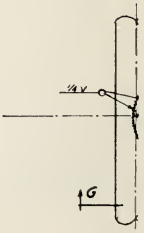


F.S. DETAIL SHOWING FIELD CONNECTION OF 8"ϕ STANDARD PIPES USED FOR TOP BRACING.

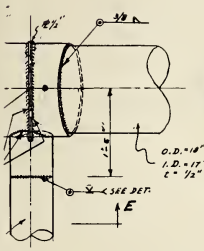
·SEC

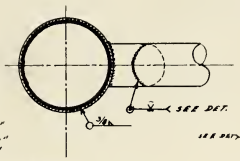


·TOP VIEW OF JOINT "a"·

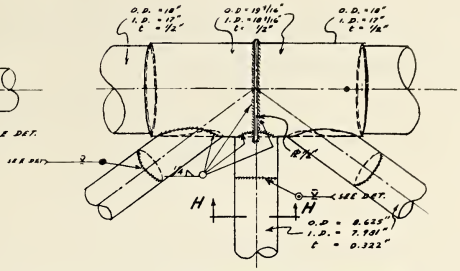


·TOP V



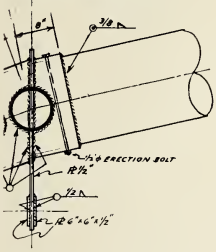


·SECTION F-F·

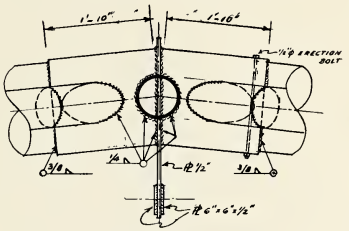


VIEW OF JOINT "C".

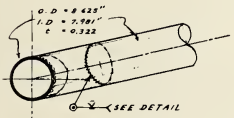
· TOP VIEW OF JOINT "E".



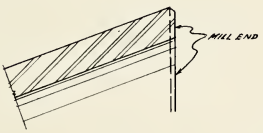
CTION E-E·



· SECTION H-H·

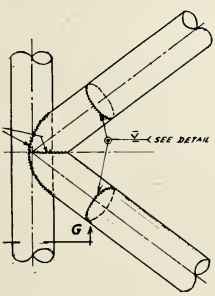


SECTION G-G·



· TYPICAL F.S. DETAIL SHOWING END OF PIPE ARCH MEMBERS·

· TYPICAL F.S. DETAIL SHOWING PIPE TO PIPE CONNECTION·



· VIEW OF JOINT "c".

NOTE: ALL DETAILS IN SCALE 1"=1'-0" UNLESS NOTED OTHERWISE.

Figure 74

"For arch and top bracing members, pipes are used as being most economical for compression stresses. They have a most pleasing appearance, have far less surface to paint and maintain, do not have so much wind load on them and they are easy to assemble and connect. Interior face of pipes shall be coated and sealed to prevent rust as hollow space in girders.

"Round bars with upset ends are used as hangers. Standard clevises, as best looking, are used for end connections.

* * *

"Since most economical sections for compression and tension members are used, there is a considerable saving of steel. Still more economy is achieved by the simpler way of erection because falsework is needed only at points of stiffening beam splices. It is strong enough to carry the crane necessary for erection of the arch. Erection of the arch members and top bracing will be simple using light falsework to support arch members so that correct shape of arch is secured before field welding is made. No camber for this type of bridge is necessary and connections shall be made with provision that under dead load the stiffening beams are straight.

"Some small items, such as floor drains and roadway connections, are not shown. Since the amount of steel used for them is very small and they are the same for both bridge designs, it was not considered necessary for estimating and comparing the cost of the two bridges.

"The system of this bridge was selected not only for its economical advantages, but also for its appearance. It is believed that a bridge is not only a means to cross an obstacle but it is also a construction showing the skill and ability of modern engineering. The simpler the lines showing the members of construction, the more impressive is the structure.

"It is also believed that a truss bridge with wide and numerous web members gives a heavy appearance and also obstructs the view of travellers crossing the bridge. However, it seems to be correct to show them that they are on a bridge which is hard to achieve as with bridges with roadway above the construction. From this point of view, the designed bridge satisfies both requirements: it shows its being there in a casual way, and the thin hangers do not prevent the travellers from having an unobstructed view. The railings are omitted because the stiffening beams are sufficiently high and this helps to make the appearance simple and clear. This

type of bridge with its uniform arch, hangers, and floor supporting construction shows the play of stresses in its functional form even to the layman and only a suspension bridge can compete with it in this respect."

The weight of the structural steel including hangers, clevises, and pins amounts to 276,022 lb. The shop welds weigh 1496 lb. and the and 32,230 lb. of reinforcement bars.

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